# SURVEYING

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# Surveying

## BOOK I

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CHAIN SURVEYING

LEVELING

COMPASS SURVEYING

320

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# CONTENTS

NOTE.—This book is made up of separate parts, or sections, as indicated by their titles, and the page numbers of each usually begin with 1. In this list of contents the titles of the parts are given in the order in which they appear in the book, and under each title is a full synopsis of the subjects treated.

LEVELING—(*Continued*)

## LEVELING—(*Continued*)



## COMPASS SURVEYING

# CHAIN SURVEYING

Serial 3066–3 Edition 1

## CHAINING

### INTRODUCTION

**1. Definition.**—Surveying is the art of determining the relative positions of points and lines on the earth's surface. By the term *the earth's surface* is meant all that part of the earth that can be explored; therefore, the term includes the bottoms of rivers and seas and the interiors of mines, as well as the more accessible points on the actual surface. From the information obtained by the methods of surveying, the locations, sizes, and areas of objects can be found either graphically or by calculation.

**2. Classification.**—Surveying may be classified, according to its purpose, as land surveying, topographic surveying, hydrographic surveying, mine surveying, etc. The general principles employed in all these classes of surveying are the same; but, according to the methods and instruments used, surveying is divided into three branches, as follows:

1. *Chain*, or *linear, surveying*, in which no other measuring instrument is employed than a tape. In former years, the chain was used for measuring distances, but it has been almost entirely superseded by the tape.

2. *Angular surveying*, in which use is made of both angle-measuring and distance-measuring instruments.

3. *Leveling*, in which the elevations of points or the vertical distances between two or more points are determined.

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**3. Instruments Used.**—Linear surveying without the aid of angle-measuring instruments is practiced only in approximate work, such as in surveys involving comparatively small areas of inexpensive land, but the measurement of distances constitutes an important part of angular surveying. Therefore, the methods of measuring distances here described include also those employed in angular surveying. The instrument used for measuring distances is generally the tape and occasionally the chain. In addition, use is made of plumb-bobs, range poles, and marking pins to facilitate the work.

## DESCRIPTION OF INSTRUMENTS

### CHAINS

**4. Description.**—A chain, Fig. 1, is composed of links of steel or iron wire, each two adjacent links being connected by small rings. The best chains are made of No. 12 steel wire and have all joints in the links and rings brazed in order to prevent their opening when a pull is applied to the chain. Some chains have two, and some three, rings between adjoining links. At the ends of the chain are handles, usually made of brass, which are attached to the chain by means of swivels having nuts and threads. Each handle forms part of the end link, the length of the chain extending to the outer edges of the handles; by means of the swivels, the length of the chain can be adjusted. Chains are classified as engineers' chains and surveyors' chains.


Fig. 1

**5.** The engineers' chain is 50 or 100 feet long. In each foot there is one link; that is, one link and the rings on one end between that link and the next make a foot. Every tenth link is marked with a brass tag to indicate its distance from the nearer end of the chain. Thus, in the 100-foot chain, shown in Fig. 1, the tags that are 10 feet from either end of the

chain are marked |, those at 20 feet | |, those at 30 feet | | |, and at 40 feet | | | |. At 50 feet, which is the middle of the tape, a plain tag, usually of different shape from the others, is used. Therefore, when a measurement is greater than half the length of the chain, the tag marked | | | | next beyond the center will represent 60 feet; and then, continuing in the same direction, tag | | | will be 70 feet; tag | |, 80 feet; and tag |, 90 feet. Hence it is important to observe on which side of the 50-foot point the tag is, in order to get the correct reading.

Engineers' chains were formerly used on all kinds of surveys where the foot was the unit of measurement, but they have been generally superseded by the steel tape.

**6.** The surveyors' chain, often called Gunter's chain, from the name of its inventor, is 66 feet, or 4 rods, long. It is divided into 100 links and, consequently, the length of a link and the connecting rings is .66 foot or 7.92 inches. This type of chain was used in all old United States land surveys, but now specially graduated steel tapes are employed instead. Whenever the word *chain* occurs in a deed, lease, or other legal document, it is understood to mean a surveyors' chain of 66 feet. The advantages of the 66-foot chain as a unit in land surveys are that there are 80 chains in 1 statute mile, and that areas expressed in square chains can be changed to acres by simply moving the decimal point one place to the left, since there are 10 square chains in 1 acre.

**7. Folding a Chain.**—A chain may be folded either from one end or from the center. To fold it from the center, take the middle pair of links together in the left hand, grasp the third pair of links from the middle with the right hand and fold the second and third pairs of links across the middle links and nearly parallel to them; then grasp the fifth pair and fold the fourth and fifth pairs across and nearly parallel to those already in place; proceed in the same way until the end is reached. The chain should then be secured by a cord or strap around the centers of the links, as shown in Fig. 1. A chain may be folded from one end in a similar manner.

### TAPES

**8. Classification.**—Tapes for measuring are usually made of steel, although cloth tapes and so-called metallic tapes are sometimes used. Since the cloth tape stretches easily and shrinks when wet, it is of little value in surveying. *Metallic tapes* are composed of linen with fine brass threads woven in lengthwise to reduce the stretching. They are made in lengths of 25, 50, 75, and 100 feet and are usually graduated to feet, tenths, and half-tenths of a foot. Their use is limited to short measurements where great accuracy is not required.

*Steel tapes* are ribbons of steel, varying in width from $\frac{1}{8}$ inch to $\frac{1}{2}$ inch, and are obtainable in various lengths from 1 yard to

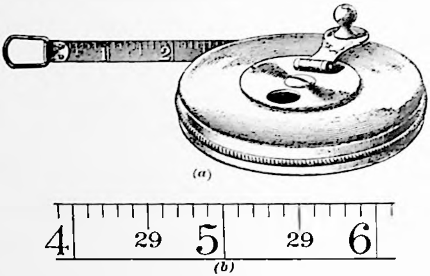


Fig. 2

1,000 feet. They are graduated in various ways, depending on the purposes for which they are used.

**9. Pocket Tapes.**—Where many short measurements are taken, a thin tape, 25 feet, 50 feet, or 100 feet long and graduated to feet, tenths of a foot, and hundredths of a foot throughout its length, is most convenient. Such tapes are usually a little less than $\frac{3}{8}$ inch wide and are enclosed in a hard-leather case with a folding crank for winding up the tape. A tape in its case is shown in Fig. 2 (*a*); it can be conveniently carried in a pocket. The graduations are etched on the tape, so that it winds easily. Often the number of the preceding foot is marked on the tape at intervals of a tenth of a foot, as illus-

trated in Fig. 2 (*b*), which shows part of a tape between 29 and 30 feet from the end. The zero point is generally at the end of the tape, but it is sometimes at the end of the small ring. Before the tape is used, the location of the zero mark must be determined.

The small ring on any tape serves for attaching a handle. A metal handle is best, but a rawhide strip tied through the ring is suitable and gives a good grip. The inner end of the tape is held on the reel by a small pin which fits in a hole in the ribbon. An additional length of ribbon is provided beyond the last graduation so that a small part remains coiled after the graduated part has been unwound.

**10.** Band Chains.—For general work it is common to use 100-foot tapes of greater thickness than the pocket tapes described in the preceding article. These tapes are graduated every 5 or 10 feet, with the first and last intervals subdivided


Fig. 3

into feet, and the first and last feet into tenths of a foot. Sometimes the tapes are marked every foot for the entire length. These heavier tapes are usually between $\frac{1}{5}$ and $\frac{1}{4}$ inch in width and are commonly called *band chains*. The graduations are generally marked on small sleeves of brass or copper soldered on the tape or on Babbitt metal brazed on both sides of the tape. In Fig. 3 is shown the graduation 56 feet from the end of a tape. Sometimes small brass or copper rivets are used to mark the feet and every 5-foot or 10-foot mark is numbered. Rivets are not so good as sleeves, since the holes for the rivets weaken the tape. On band chains the last graduations are usually some distance from the ends of the tape, and metal or rawhide handles are attached through the end rings of the tape. When long lines are to be measured and the slope of the ground permits, 300-foot or 500-foot band chains can be used to advantage. The band chains used in land surveying are graduated to links and are 1, 2, 5, or 8 chains in length. Usually, each 5-link mark is numbered.

Band chains are wound on reels of metal or wood. A typical metal reel is shown in Fig. 4, and a wooden reel is shown in Fig. 5. The sides of the guides *a* are just far enough apart to permit one width of the tape to pass between them. In order to hold the tape on the wooden reel shown in Fig. 5 after it has been wound up, it is tied near the ends with cords or leather strips *b*.

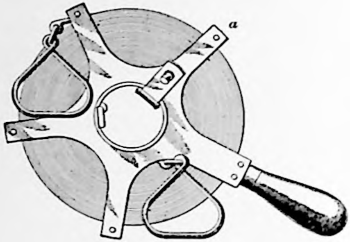


Fig. 4

**11. Handling Tape.** When a tape is being used, it is unrolled to its full length and detached from the reel. For convenience in carrying the wooden reel when it is empty, the pin *c*, Fig. 5, can be removed and the reel can then be folded. There are many other types of reels, some of which have arm straps to assist in holding the reel when long tapes are being wound. On some reels the sides of the guides are farther apart and permit the tape to spread on the reel in winding.

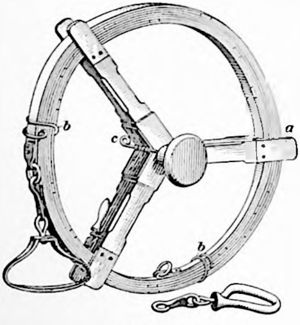


Fig. 5

When only part of the tape is being used, or when the tape is being carried from place to place for measurements, it is usually inconvenient to keep the tape on the reel and to wind and unwind it continually as the occasion requires. If it is not advisable to leave the tape spread out, an easy way to hold the tape when off the reel is to coil it somewhat like a rope. To do this, take the zero end of the tape in the left hand with the graduated face

upward and, without twisting the tape, run it through the fingers of the right hand until the 5-foot mark is reached. Then, always keeping the graduations upward, place the 5-foot mark directly over the 0 mark in the left hand, run more of the tape between the fingers of the right hand, and place the 10-foot mark over the 5-foot division. Continue this operation for the entire length of the tape, placing each 5-foot division over the preceding one. Then when the tape is held in the hand at the 5-foot graduations, it will fall in the approximate form of a figure 8. The tape can also be held conveniently at the center of the 8 as shown in Fig. 6. The tape can easily be uncoiled by releasing the loops one at a time

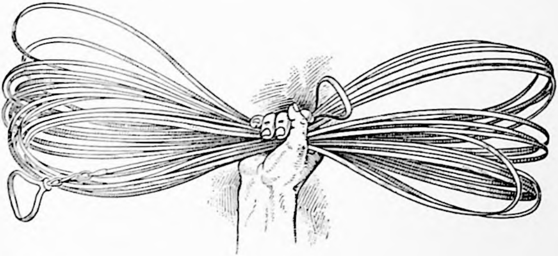

Fig. 6

as required. Several coils should not be dropped at once, since then the tape is likely to become tangled.

Cloth tapes should be used only for very approximate work and in places where they can be kept clean and dry. They are easily broken and need careful handling.

Pocket tapes may be employed in wet or dirty locations. But, when exposed to the least bit of moisture, they should be wiped perfectly dry and then rubbed with some light oil, which will not stain the hands. They are fragile and break very easily; if run over by a vehicle or struck by an axe or stake, they are quite likely to be broken.

Band tapes of suitable material can be injured only by gross carelessness. If they are pulled while looped, they are likely to

kink and break. They may be used all day in the rain and mud, and they only need to be hung where they will dry during the night. Still many surveyors prefer to clean them.

Pocket and band tapes are mended by attaching an extra piece at the break with two rivets on each side of the joint or by soldering a sleeve over the broken portion. Care should be taken that the proper distance between the two adjoining marks is maintained and that the tape is kept straight. For splicing a tape in the field, for a few hours or even a day or two, a device consisting of a sleeve with a setscrew at each end is sometimes used. Since repaired instruments are seldom as good as before they were damaged, the utmost care should be used to avoid accidents.

**12.** Although tapes have practically supplanted chains, the word *chaining* is still commonly used to describe the operation of measuring distances. The methods employed in measuring are substantially the same, no matter what style of tape is used, and, therefore, the term *tape* will here be used in its general meaning unless otherwise stated. Measurements are usually recorded in feet and decimals, but those taken with a tape graduated to links are given in chains and links.

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### ACCESSORIES

**13. Plumb-Bobs.**—A plumb-bob can be made by attaching any kind of a weight to the end of a string; when the weight is allowed to hang freely, the line of the string points toward the center of the earth and is vertical, or *plumb*. Typical plumb-bobs for surveying work are shown in Fig. 7. The type commonly used is shown in (*a*); it consists of a brass body *a*, into which are screwed the brass cap *b* and the steel point *c*. Part of the bob is shown in cross-section to illustrate the method of inserting and fastening the string. In view (*b*) is shown a steel bob which is suitable for work near walls or other surfaces and which has also the advantage that there is not so much surface exposed to the wind. The bob shown in (*c*) is made of iron, and is solid except for a hole in the top to allow the string

to be fastened. This style of bob is used only for very rough work. In all types of bobs, it is important that the bob should be exactly centered on a line through the point and the hole in the top where the string is inserted. The weight of bobs varies from about 8 ounces to 3 pounds. Light plumb-bobs sway in the wind while heavy ones are inconvenient to carry. A weight of 1 or $1\frac{1}{2}$ pounds is preferred by most surveyors.

The cord for plumb-bobs should be fine and light but should be strong enough to resist a pull in excess of the weight of the bob. A special plumb-bob string is manufactured. For very accurate work piano wire is often employed.

**14. Range Poles.**—Wooden or steel poles or rods called range poles are placed at points to which measurements or

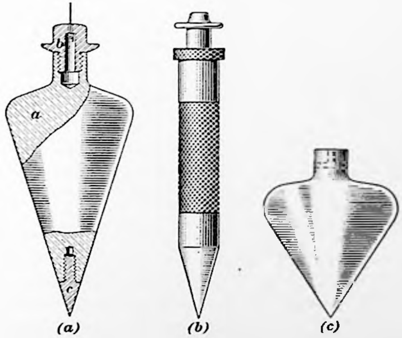


Fig. 7

sights are taken when the point cannot be seen otherwise. When made of wood, they are usually 8 or 10 feet long, and are either circular or octagonal in section with a diameter of about 1 inch. A pointed iron shoe is attached to the lower end so that the rod can be stuck in the ground and will then stand erect without being held. Steel rods are 6 or 8 feet long and are usually hexagonal in section with a thickness of about $\frac{1}{2}$ inch. They are pointed at the lower end so that they can be

stuck in the ground in any desired position. Both wooden and metal poles are divided into foot-long spaces painted alternately red and white to make them visible at a long distance. Often a piece of cloth is tied at the top of the pole to make the pole more easily seen and, therefore, range poles are sometimes called *flagpoles*, or simply *flags*, and the men who handle them are called *flagmen*. If it is desired to sight past the pole, the flagman should stand to one side; otherwise, he should stand behind the pole. Poles should be held as nearly plumb as possible. If the pole is not stuck in the ground it can be kept vertical by resting the point on the ground and balancing the pole between the fingers; if the pole is secured in the ground, it can be made to stand vertical by shifting it until it is parallel to a plumb-bob string held near it

**15. Marking Pins.**—For marking temporarily the ends of measurements, where only the distance between certain points is needed, marking pins are used. These are slender rods of iron or steel about 12 or 14 inches long, pointed at one end and bent into a ring at the other. They can be stuck in the ground where needed. When used in grassy or weedy ground, pieces of cloth are tied to the rings so that the pins can be readily found.

**16. Stakes and Monuments.**—When the points set in chaining are to be used later, their location should be fixed. For permanent points, concrete or stone monuments are commonly placed in the ground with a cross mark chiseled in the top to indicate the exact point. If the location of the point is needed for a short time only, wooden stakes are driven and left in the ground. A definite point can be marked by a small tack in the top of the stake or by a small cross in pencil. Stakes must be suitably marked for identification, and for this purpose a specially prepared crayon, called *keel*, is commonly used. Often, to aid in identifying a point, an additional stake, called a *witness stake*, is placed near the point.

## MEASURING DISTANCES

### METHODS OF CHAINING

**17. Distances.**—Unless otherwise stated, when the distance between two points on the earth's surface is given, as on a map, the horizontal distance is meant. For example, if in Fig. 8 the surface of the ground is represented by the line $AB$, the distance from $A$ to $B$ is represented on a map as the length $AB'$ along the horizontal line $AX$, the line $BB'$ being vertical. Therefore, in surveying, the horizontal distance between $A$ and $B$ must be determined.

**18.** Distances in surveying must be measured in a straight line, as well as horizontally, because a straight line between two points is shorter than any other line and always has the same length. Therefore, when a distance is measured, it is important to see that there are no bends or twists in the tape; if the measurement is made in several parts, the intermediate points should lie on the straight line between the ends.

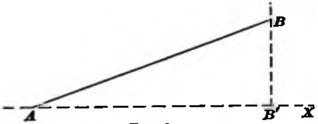

FIG. 8

**19. Measuring on Level Ground.**—Two men are necessary for all chaining; they are called the *head chainman*, or *front chainman*, and the *rear chainman*, according to their positions in reference to the direction in which the measurement is being taken. If the tape is still on the reel, the head chainman takes the exposed end and walks ahead, while the rear chainman remains at the beginning of the line and holds the reel so that the tape unwinds. If the tape is coiled in 5-foot loops, the head chainman goes ahead with one end, while the rear chainman releases one loop at a time. A chain is undone by holding both handles in one hand and throwing it forcibly with the other hand so that the links will be free from each other. The chain should be thrown in the direction opposite to that

in which the measurement is to be made, so that when the head chainman takes one handle and drags the chain ahead, the rear chainman can stand at the point of beginning and straighten out any kinks in the chain as it is drawn past him.

When almost all of the tape or chain has been drawn past, the rear chainman calls, "Chain," and the head chainman stops and straightens out the tape. The rear chainman holds his end of the tape firmly at the stake or pin that marks the beginning of the measurement, and the head chainman holds a stake or pin at his end of the tape, keeping his body to one side of the line. The tape is then pulled taut, and the rear chainman, sighting along the line from the point he occupies to a range pole or some other object that determines the direction, signals to the head chainman to move to the right or left as required, until his stake or pin is on line. When the tape is taut and straight and the stake or pin held by the head chainman is on line, the rear chainman calls, "All right." The head chainman then places the stake or pin in the ground and replies, "All right." When the line to be measured is longer than one tape length, the operation is repeated as often as required.

The length of any line can thus be determined by a series of measurements similar to that just described. For each measurement the rear chainman holds his end of the tape at the point marking the end of the last preceding tape length, and lines in the head chainman, who sets the next point ahead. When the end of the line is reached, the head chainman walks on past that point with the tape until the rear end is at the last stake or pin. He then returns, and the men measure the distance from the last pin to the end of the line. Sometimes the head chainman holds his end of the tape at the end of the line and the rear chainman takes the reading opposite the last pin, it thus being unnecessary for the head chainman to walk past the end of the line. The total length of the line will be equal to a number of full tape lengths, plus a single partial tape length at the end. In order to avoid mistakes in the number of full tape lengths, it is important to have a system of counting them by which the result can be readily checked.

**20.** When a stake is placed at each tape length, it is numbered so that the distance from the beginning of the line is shown. Thus the starting point is called 0, and when the head chainman has placed the stake at the end of the first tape length, he marks it with the figure 1. When the rear chainman arrives at this stake to start the second measurement he calls, "One;" the head chainman sets the next stake and, after announcing, "Two," so that the rear chainman can correct him if it is not the proper number, he marks the stake 2. This operation is repeated at each measurement and thus the distance to the last stake is always known. Then the length of the partial measurement at the end of the line can be added to the distance represented by the number of full tape lengths. For example, if the stake at the end of the last full 100-foot tape length is marked 8 and the distance from this stake to the end of the line is found to be 36 feet, the length of the line equals $8 \times 100 + 36 = 836$ feet.

**21.** If chaining pins are used to mark the ends of measurements, it is convenient to use a set of eleven pins. When the head chainman starts, he carries ten pins, leaving the eleventh to mark the beginning of the line; or, if the starting point is otherwise located, the eleventh pin is left with the rear chainman. The rear chainman pulls out each pin after the measurement from it has been taken. When ten pins have been set, the head chainman calls, "Tally." He then receives from the rear chainman ten pins with which to start again, the eleventh pin being left in the ground to mark the end of the tenth tape length. Each tally should be recorded, either by writing or in some other convenient manner. By counting the number of tallies and the number of pins in the possession of the rear chainman, exclusive of the one in the ground, the distance to the last pin is calculated; then the distance from the last pin to the end of the line is added.

EXAMPLE 1.—A 100-foot tape is used and there have been 3 tallies, the rear chainman has 6 pins, and the distance from the last pin to the end of the line is 37.8 feet. Find the length of the line.

SOLUTION.—Each tally represents 10 full tape lengths and each pin held by the rear chainman represents 1 full tape length. Hence, the total

number of full tape lengths equals $3\times10+6=36$ and the distance to the last pin equals $36\times100=3{,}600$ ft. Since the distance from the last pin to the end of the line is 37.8 ft., the length of the line equals $3{,}600+37.8=3{,}637.8$ ft. Ans.

EXAMPLE 2.—What distance has been measured with a tape, 1 chain long, if 4 tallies have been recorded, the rear chainman has 7 pins, and the last partial measurement is 47 links?

SOLUTION.—The 4 tallies represent $4\times10$ or 40 ch. and the 7 pins represent 7 ch.; each link is .01 ch. and 47 links are .47 ch. Hence, the total distance equals $40+7+.47=47.47$ ch. Ans.

**22. Reading Tape for Partial Measurements.**—In measuring a line by chaining, it is usually necessary at the end to take a measurement that is not a full tape length. In many cases, intermediate partial measurements are also required because of the impracticability of setting a point at the full tape length; for instance, the point may come in a stream or other inaccessible place. It is also common to locate various objects near a line by short measurements from points along the line. These points are seldom at the end of a full tape length. In general work, almost all measurements are partial tape lengths.

The method of measuring a distance less than a full tape length is the same as for a tape length, except that the rear chainman holds some graduation at his point instead of the end of the tape. When a tape divided in links is used, the measurement is usually taken to the nearest link, and the number of links can be readily determined by counting from one of the 5-link graduations. For example, if the head chainman holds the zero end of the tape, and the point occupied by the rear chainman is near the third link past the 40-link mark the measurement is $40+3$, or 43, links. If tenths of a link are desired, they can be estimated by eye; the value will be close enough, since these tapes are never used in accurate work.

If the tape is graduated to feet, tenths, and hundredths throughout its length, the measurement to hundredths of a foot is read directly on the tape; if desired, thousandths of a foot can be estimated with a little practice. If a band chain, graduated to feet, and the end foot to tenths, is employed, the

rear chainman holds some foot mark at his point so that the end of the measurement falls within the end foot held by the head chainman, as shown in Fig. 9, where $A$ is the end of the line to be measured. Then the head chainman reads the number of tenths, and, if desired, estimates the hundredths, from the end of the tape to the end of the measurement; and this value is subtracted from the number of feet indicated by the graduation held by the rear chainman. For example. suppose the rear chainman holds the 54-foot graduation at his point and the other end of the measurement falls between 7 tenths and 8 tenths at the zero end of the tape, as in Fig. 9. If the head chainman estimates that the reading is 73 hundredths, the length of the measurement is recorded as $54-.73$, or 53.27 feet.

**23. Stations.**—Important points on a survey line are called *stations*. These may be at the ends of tape lengths, as mentioned in Art. 20, or at such definite points as may be

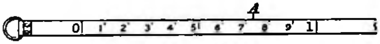

FIG. 9

required for future reference. In the latter case they are marked permanently by stakes or monuments. Stations may be identified by giving them either letters or numbers, but the method generally used for referencing is to number stations according to the hundreds of feet they are distant from the starting point of the survey; these distances are measured along the line of the survey. Thus, the starting point is Station 0, a point 400 feet from the start is Station 4, and a point 1,200 feet from the start is Station 12. A point whose distance from Station 0 is not a multiple of 100 feet is marked by the number of the 100-foot station immediately preceding, plus the distance from that station to the point in question. For example, a point between Stations 5 and 6, and 47 feet from 5, is marked 5+47 and is called Station 5 plus 47. Its distance from Station 0 is 547 feet.

EXAMPLE.—Find the distance in feet between Stations 3+76.4 and 6+13.1.

SOLUTION.—Sta. 6+13.1 is 613.1 ft. from the beginning of the line and Sta. 3+76.4 is 376.4 ft. from the start. Hence, the distance between the given points equals $613.1-376.4=236.7$ ft. Ans.

**24. Measurement on Sloping Ground.**—Very often the two points on the ground between which a measurement is taken, as $A$ and $B$ in Fig. 10, are at different elevations. One method of determining the horizontal distance from $A$ to $B$ is to measure between $A$ and $B$ with the tape held horizontally.

In taking a horizontal measurement on sloping ground, three things must be considered: (1) The tape must be horizontal, as represented by line $A''B$ in Fig. 10; (2) the point $A''$, indicating the end of the measurement on the tape, must be vertically above point $A$ on the ground; (3) the pull on the tape should be such that the stretch will be equal to the shortening of the length due to the fact that a tape or chain suspended only at the ends hangs in a curve, or *sags;* the straight-line distance between the ends is, therefore, less than the length of the tape.

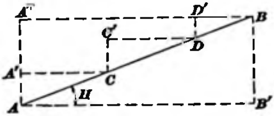

FIG. 10

It is a common error to hold the down-hill end of the tape too low. To aid in judging when the tape is horizontal, lines on nearby buildings may sometimes be used. In precise work, a hand level is used for that purpose.

The end of the measurement is best transferred to the ground by means of a plumb-bob. The plumb-bob is held so that its point is close to the ground, but not touching it; when the tape is taut and horizontal and the end properly lined, the plumb-bob string is released by the chainman and the pin or stake is placed where the point of the bob strikes the ground. If the wind is blowing, it is necessary to stand so as to protect the bob from its force. The swingingof the bob can be stopped by allowing it to touch the ground and raising it slightly or by a slight counter movement of the hand holding the string.

Measurements are more easily made down-hill because then the rear chainman can hold his end of the tape firmly and the

head chainman can pull steadily; when measuring up-hill, the rear chainman must hold a plumb-bob over his point, and it is difficult to keep it steady while the head chainman is pulling on the tape. When measuring across a gully, much time will be saved if a measurement can be taken between points at the same elevation on opposite slopes.

**25.** If the difference in elevation between two points a full tape-length apart is too great to permit the tape to be held horizontal, the measurement can be made by *breaking chain.* Suppose that, in Fig. 10, $A$ and $B$ are a tape length apart, but

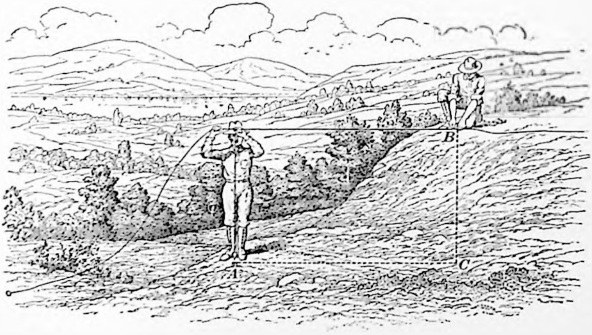


FIG. 11

$B$ is much higher than $A$. In such a case, the distance $A\,B$ is measured in sections by establishing intermediate points, such as $C$ and $D$, as far apart as possible but preferably at distances that are multiples of 10 feet, as 40 feet or 70 feet. Then distance $A\,B'$ is the sum of distances $A'C$, $C'D$, and $D'B$. However, it is sometimes more convenient, especially when stakes are driven, to select the points on the line, as $C$ and $D$, and then measure the distances $A'C$, $C'D$, and $D'B$. The method of measuring by breaking chain is illustrated in Fig. 11. The points on the ground, between which the measurement is taken, are $A$ and $B$. The rear chainman holds a point of the tape at $B$ and the head chainman holds a plumb-

bob over $A$. The distance recorded on the tape is the horizontal distance $A\,C$.

When a line is being measured by breaking chain, mistakes in reading the tape and adding the distances will be avoided if the rear chainman holds the zero point of the tape at the beginning of the measurement, and then at each intermediate point, he holds the same graduation as the head chainman had when the preceding distance was measured. This operation is repeated until the required distance is found or the end of the tape is reached. It is then unnecessary to record the partial measurements prior to the end of the line. When pins are used to mark the intermediate points in measuring by breaking chain, the rear chainman should keep only those pins that mark the full tape lengths; the others should be given back to the head chainman as soon as they are removed from the ground.

**26.** If the ground is very steep, the horizontal measurements must be very short. Therefore, when the slope is uniform, it is better to measure the distance along the surface as if the ground were level and to determine the angle of inclination of the ground. (The method of measuring this angle will be described in another Section.) Then the horizontal distance can be calculated by multiplying the inclined distance by the cosine of the angle of inclination; for example, in Fig. 10,

$$A\,B' = A\,B \cos H \qquad (1)$$

Sometimes the difference in elevation, or the vertical distance, between the two points is known. Then it is unnecessary to measure the angle of inclination, the horizontal distance being found from the inclined distance and the vertical distance as follows: In the right triangle $A\,B'B$, Fig. 10, $\overline{A\,B}^2 = \overline{A\,B'}^2 + \overline{B'B}^2$, or $\overline{A\,B'}^2 = \overline{A\,B}^2 - \overline{B'B}^2$. Hence,

$$A\,B' = \sqrt{\overline{A\,B}^2 - \overline{B'B}^2} \qquad (2)$$

EXAMPLE 1.—A distance, measured along the ground, is 118 feet and the angle of inclination is 18° 12′. If the line is to be plotted on a map, what length would be used?

SOLUTION.—Distances on maps are horizontal and the horizontal projection of an inclined line is found by formula **1**. Then with Fig. 10 as a

diagram, $AB=118$ and $H=18° 12'$. Therefore, $AB'=118\times\cos 18° 12'$ $=118\times.950=112.1$ ft. Ans.

EXAMPLE 2.—If the difference in elevation between two points $A$ and $B$ is 60 feet and the distance $AB$ measured along the ground is 170.9 feet, what is the horizontal distance from $A$ to $B$?

SOLUTION.—The hypotenuse and one leg of a right triangle are known and, therefore, the length of the other leg can be found by formula 2. With reference to Fig. 10, $AB=170.9$ and $B'B=60$; then $\overline{AB}^2=29{,}207$, $\overline{B'B}^2=3{,}600$, and $AB'=\sqrt{29{,}207-3{,}600}=\sqrt{25{,}607}=160.0$ ft. Ans.

#### EXAMPLES FOR PRACTICE

1. The distance between two points, measured along the ground, is 216.3 feet. If the ground slopes at an angle of 7° 39′ to the horizontal. what length should be used on a map in locating one point from the other?
Ans. 214.4 ft.

2. Find the horizontal distance between two points if the inclined distance between them is 411.8 feet and the difference in elevation is 81.5 feet.
Ans. 403.7 ft.

3. A line was measured with a 200-foot tape; there were 2 tallies, the rear chainman had 3 pins, and the distance from the last pin to the end of the line was 19.4 feet. Find the length of the line. Ans. 4,619.4 ft.

4. A 2-chain tape was used to measure a line; 3 tallies were recorded, the rear chainman had 5 pins, and the distance from the last pin to the end of the line was 81.1 links. What was the length of the line?
Ans. 70.811 ch.

#### ERRORS AND CORRECTIONS

**27. Sources of Errors.**—Errors in chained distances are of two classes: (1) errors that are due to faulty chaining and to natural conditions; and (2) mistakes in reading or recording measurements. The chief sources of errors of the first class are: (*a*) having the forward end of the tape off line or not having the tape horizontal; (*b*) having a pull different from that required to compensate for the effect of sag and of the wind; (*c*) careless plumbing; (*d*) incorrect length of tape, and (*e*) variation in temperature. Errors in reading or recording measurements may be caused by: (*a*) using the wrong zero point on the tape; (*b*) reading the wrong foot-mark, as 46.8

instead of 45.8; (*c*) reading in the wrong direction from a graduation, as 38 instead of 42 or 38.3 for 37.7; (*d*) reading the tape upside down, thus mistaking some figure, perhaps a 6 for a 9 or vice versa; (*e*) transposing figures in recording, as 71.23 instead of 72.13; (*f*) the note keeper misunderstanding the reading called by the chainman; and (*g*) mistakes in counting the full tape lengths.

**28. Preventing Mistakes.**—The zero point of the tape should be carefully located before starting any measurement. Mistakes in reading the wrong foot-mark may best be avoided by looking at the foot-mark on each side of the reading; then it is certain between which two feet the reading must be. Similarly the possibility of reading in the wrong direction from a graduation is removed by noting the graduations on both sides of the reading. The transposition of numbers in recording is purely a mental slip and can be prevented only by exercising great care.

Except when the head chainman is keeping the notes, he should call all readings distinctly and in such terms that there can be no doubt of the value. For example, the chainman calls "thirty nine three," meaning 39.3, but the note keeper might write 30.93 if readings are taken to hundredths; also, "thirty nine three" at a distance might sound like "thirty-nine feet." Hence, 39.3 should be called "thirty-nine, thirty." The note keeper should always call back the reading for the chainman's approval before he records it, and if possible should repeat it differently. Thus, if the chainman gives a reading as "twenty five," the note keeper could answer "twenty and a half" so that if the chainman meant 25.0 or 20.05 the difference would be readily noticed. In order to avoid mistakes of this kind, the correct way to call the following values is given: 20.05, "twenty, naught five;" 20.5, "twenty fifty" or "twenty and a half;" 25.0, "twenty-five, naught" or "twenty-five feet, even." Mistakes in counting the tape lengths should not occur if the pins are handled correctly or if the stakes are marked and the numbers called as previously explained.

**29. Alinement.**—The error in length due to the tape being not quite horizontal or not quite in the proper direction is very small. If one end of a 100-foot tape is held 1 foot out of line or 1 foot lower than the other end, the error in a tape length is about $\frac{1}{16}$ inch. Therefore, too much time should not be wasted in lining in the intermediate pins or stakes when measuring the line. However, if it is desired to take a measurement from an intermediate point on a line, that point should be lined in accurately. For example, the point *C*, Fig. 12, set on the line *A B* for the purpose of measuring distance *C D*, should be located carefully.

**30. Sag and Pull.**—The pull required to compensate for the effect of sag depends on the length and the cross-sectional area of the tape. In general, a pull of about 10 pounds is best. The pull can be measured accurately by means of a spring balance attached to the handle at one end of the tape. The proper pull for a given tape may be determined by measuring between two points that are a known distance apart. This refinement is necessary only in very accurate work. When a chain is used, it should be taut, but should not be pulled with so much force that tho rings stretch. An experienced chainman can apply the proper pull by judgment.

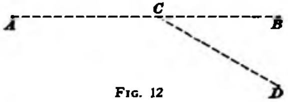

Fig. 12

**31. Correction for Erroneous Length of Tape.**—The length of a chain or a steel tape is altered by wear and distortion, and by changes in temperature. The length is increased when the temperature rises and decreased when the temperature falls. The length of a steel tape is specified at a given temperature, the change in the length of a 100-foot tape for a difference in temperature of 15 degrees Fahrenheit (15° F.) being about $\frac{1}{8}$ inch, or .01 foot. In accurate surveying the variation due to temperature must be considered.

**32.** The permanent change in length in a 100-foot tape due to wear and distortion is sometimes as much as $\frac{1}{2}$ inch. This shows the necessity of handling the tape carefully; if it is

caught, as in a stump, in a rail joint, or in any other way, pulling from the ends may stretch it. The length of a tape should therefore be tested frequently, either by comparing it with a tape of standard length or by measuring between two points a known distance apart. It is advisable to have two such points marked permanently on a smooth pavement, curb, or some other convenient place. The length of a chain can be adjusted to the standard length by means of the nuts or swivels on the handles.

After a line has been measured, it is often found that the length of the tape is in error; the true length of the line must then be determined. For example, suppose it is required to calculate the true length of a line that was measured as 634.2 feet with a 100-foot tape that was afterwards found to be .05 foot too long. Since a 100-foot tape was used, there were $\frac{634.2}{100}$, or 6.342, tape lengths. But each tape length was .05 foot more than 100 feet. Hence, the true measurement was $6.342 \times 100 + 6.342 \times .05$. In general, when the tape is too long, the correct distance equals the measured distance plus the number of tape lengths times the amount by which the tape exceeds its proper length. This statement may be expressed in a formula.

Let $T$ = true, or correct, distance, in feet;
$M$ = measured distance, in feet;
$L$ = tape length, in feet;
and $e$ = error in one tape length, in feet.

Then,
$$T = M + \frac{Me}{L} \qquad (1)$$

If the tape is too short, similar reasoning will show that the correct distance is given by the equation

$$T = M - \frac{Me}{L} \qquad (2)$$

If the tape is too long, the true distance will be longer than the field measurement, while if the tape is too short, the true distance will be less than the recorded value. In using these formulas, other units, such as inches, chains, or meters, may be

employed instead of feet, but the units must be the same for all quantities in any one formula.

EXAMPLE 1.—The length of a line measured with a 50-foot tape was recorded as 1,048.4 feet. It was afterwards found that the length of the tape was 50.03 feet. What is the true length of the line?

SOLUTION.—The error $e$ in one tape length equals $50.03-50.00=.03$ ft.; $M=1,048.4$ ft.; and $L=50$ ft. Then, by formula **1**,

$$T=M+\frac{Me}{L}=1,048.4+\frac{1,048.4\times .03}{50}=1,048.4+0.6=1,049.0 \text{ ft. Ans.}$$

EXAMPLE 2.—The length of a line measured with a 66-foot tape was recorded as 19.89 chains. If the tape was $1\frac{3}{4}$ inches too short, what is the true length of the line?

SOLUTION.—Since the measurement is in chains, $e$ must likewise be expressed in chains. Then, the error $e$ in each tape length equals $1\frac{3}{4}$ in. $=1.75$ in. $=\frac{1.75}{12\times 66}$ ch. $=.0022$ ch.; $M=19.89$ ch.; and $L=1$ ch. Hence, by formula **2**,

$$T=M-\frac{Me}{L}=19.89-\frac{19.89\times .0022}{1}=19.89-0.04=19.85 \text{ ch. Ans.}$$

**33.** When a given distance is to be laid off with a tape, the length of which is in error, the corrected measurement may be calculated as follows: Suppose that a distance of 456.22 feet is to be measured with a 100-foot tape which is known to be .05 foot too long. If a tape exactly 100 feet long were used, the number of tape lengths would be $\frac{456.22}{100}$, or 4.5622. Since each tape length is .05 foot too long, the actual length of the line if measured with the erroneous tape would be $4.5622\times 100+4.5622\times .05$. When the tape is too long, the corrected measurement will therefore be obtained if, from the given distance, is subtracted the number of tape lengths times the error in each. This statement may be expressed in a formula.

Let $T$ = given distance, in feet;
$M$ = distance, in feet, to be measured with the erroneous tape, to equal the given distance;
$L$ = tape length, in feet;
and $e$ = error in one tape length, in feet.

Then, $$M = T - \frac{Te}{L} \qquad (1)$$

If the tape is too short, similar reasoning will show that the corrected measurement is given by the equation

$$M = T + \frac{Te}{L} \qquad (2)$$

If the tape is too long, the corrected measurement will be shorter than the true distance, and formula **1** applies. If the tape is too short, the corrected measurement will be greater than the given value and formula **2** must be used. As in the preceding article, other units may be employed instead of feet, provided that the units are the same for all quantities in a formula.

EXAMPLE.—A city block 400 feet in length is to be measured with a 100-foot tape that is 100.021 feet long. What distance should be laid off?

SOLUTION.—Here, the error $e$ in one tape length equals $100.021 - 100.000 = .021$ ft.; $T = 400$ ft.; and $L = 100$ ft. Then, by formula **1**,

$$M = T - \frac{Te}{L} = 400 - \frac{400 \times .021}{100} = 399.916 \text{ ft.} \quad \text{Ans.}$$

**34. Precision Required.**—No survey can be made absolutely free from errors, but the allowable error varies with the kind of work. Since the accuracy with which measurements are taken affects the cost of the survey, the work should be done to give the desired degree of precision with the least expense. For land surveys where the value of the land is about \$40 or \$50 an acre, the permissible error in chaining may be as much as 1 foot in 500 feet; but if the land is worth about \$500 an acre, the error should be less than 1 foot in 2,000 feet. The error in measurements for engineering structures is often limited to 1 foot in 10,000 feet.

If the allowable error is 1 in 500, no refinements in chaining are necessary. The error can easily be made less than 1 in 5,000 if ordinary care is taken in lining, plumbing, and correcting for large errors in the length of the tape. To obtain results in which the error is less than 1 in 10,000, allowance must be made for variations in length caused by differences in pull and in temperature.

### EXAMPLES FOR PRACTICE

1. The length of a line measured with a 100-foot tape was recorded as 1,946.2 feet. If the tape was .03 foot too short, what is the true length of the line?
Ans. 1,945.6 ft.

2. A line is measured with a 66-foot tape and its length is found to be 8.94 chains. If the tape is .4 link too long, find the true length of the line.
Ans. 8.98 ch.

3. A certain line is to be laid off with a 100-foot tape that is 99.987 feet long. If the required distance is 336 feet, what length should be laid off?
Ans. 336.044 ft.

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## FIELD PROBLEMS

**35. To Prolong a Line.**—In Fig. 13, $A B$ is a line that it is desired to prolong beyond its extremity $B$. Having marked $A$ and $B$ by vertical range poles at these points, the surveyor walks some distance back of $A$ and places himself at a point $P$ in line with $A$ and $B$, by sighting to the pole at $A$,


Fig. 13

and stepping to the right or to the left until the pole at $B$ is covered by that at $A$. He then directs a pole to be held beyond $B$ and signals to the flagman until the latter has the pole in such position that it is covered by the poles at $A$ and $B$. Let $Q$ be the point thus determined. A pole being stuck at $Q$, the surveyor moves to $A$, and, sighting along $B Q$, lines in the flagman at a point $R$ beyond $Q$. The process may be repeated and the line prolonged as far as necessary. The distances $A B$, $B Q$, $Q R$, etc., should be as long as they can be conveniently made. Steel poles are thinner than wooden poles and are therefore preferable.

**36. To Run a Line Over a Hill Between Two Points, Neither of Which Is Visible from the Other.**—The points $A$ and $B$, Fig. 14, are supposed to be invisible from each other because they are on opposite sides of a hill. A line can be

run between $A$ and $B$, and intermediate points on the line can be set, as follows: One pole is placed at $A$ and another at $B$; then two men with poles station themselves at points, as $C$ and $D$, which they judge to be in line and in such positions that the poles at $B$ and $D$ are visible from $C$ and the poles at $A$ and $C$ are visible from $D$. The man at $C$ first lines in the pole at $D$ between $C$ and $B$, and then the man at $D$ lines in the pole at $C$ between $D$ and $A$. From his new position, the man at $C$ again lines in the pole at $D$, and so on, until the pole at $C$ is in line between $D$ and $A$ at the same time that the pole at $D$ is in line between $C$ and $B$. The points $C$ and $D$ are then in line with $A$ and $B$, and any other points can be lined in between $A$ and $C$, $C$ and $D$, or $B$ and $D$.

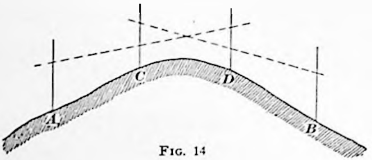

Fig. 14

**37. To Erect a Perpendicular to a Line at a Given Point.** In a right triangle, the square of the hypotenuse is equal to the sum of the squares of the legs. Therefore, since $5^2 = 3^2 + 4^2$, a triangle having sides proportional to the numbers 5, 3, and 4 is a right triangle with the right angle opposite the longest side. For example, if, in Fig. 15 ($a$), $BC = 30$ feet, $BD = 40$ feet, and $CD = 50$ feet, then $BCD$ is a right triangle

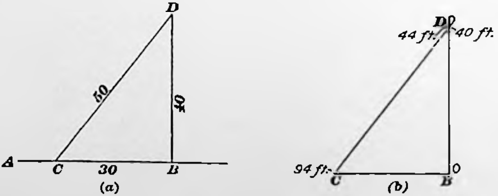

Fig. 15

and $BD$ is at right angles, or perpendicular, to $AB$. Hence, to run a line from point $B$ perpendicular to the line $AB$, locate point $C$ on line between $A$ and $B$ and 30 feet from $B$.

If one man holds the zero mark of a 100-foot tape at point $B$ and another holds the 90-foot mark at point $C$, a third, holding the 40-foot mark, can locate point $D$ at that mark by moving about until both parts of the tape are taut; for, since $BD = 40$ feet and $CD = 90 - 40 = 50$ feet, $BD$ is then perpendicular to $AB$. However, a steel tape cannot be bent to a sharp angle, as would be necessary at $D$. For this reason, the work must be done in the following manner: One man holds the zero mark at point $B$, but the second man, instead of holding the 90-foot mark at point $C$, holds the 93-, 94-, or 95-foot mark at that point. The extra 3, 4, or 5 feet is allowed to form a loop at $D$, as shown in (b), in order to avoid a sharp bend in the tape. If, as indicated in (b), the man at point $C$ holds the 94-foot mark on the point, the man at $D$ holds the 40-foot and the 44-foot marks together, with the part of the tape between 40 and 44 feet in the form of a single loop; if the 93-foot mark is held at $C$, the 40- foot and 43-foot marks are brought together at $D$ so that the length of the line $CD$ is 50 feet.

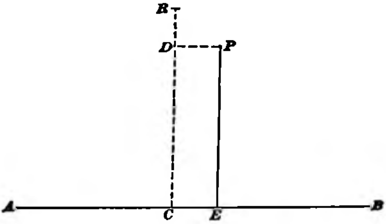

FIG. 16

The distances $BC$, $BD$, and $CD$ may be any multiples of 3, 4, and 5, but the distance $BD$ should be as long as possible to give the best indication of direction; with a 100-foot tape, the lengths 30, 40, and 50 feet are most convenient for the sides of the triangle.

**38. To Drop a Perpendicular to a Given Line From a Given Point.**—In Fig. 16, let it be required to drop a perpendicular to the line $AB$ from the point $P$. First, select by judgment a point $C$ on $AB$ where it seems that the perpendicular will meet $AB$. Then at $C$ erect a perpendicular to $AB$ by the method described in the preceding article; make

this perpendicular long enough to extend beyond $P$, as to $R$. Next, measure the distance $PD$ from $P$ to $CR$; in measuring this distance, $D$ should be on the line $CR$, but since $PD$ will

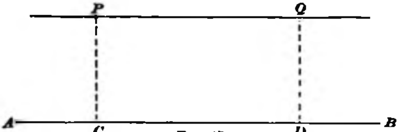


FIG. 17

be short, its direction can be determined by eye with sufficient accuracy. Finally lay off $CE$ equal to $PD$. Then $PE$ is the desired perpendicular.

**39. Through a Given Point, to Run a Line Parallel to a Given Line.**—Let $AB$, Fig. 17, be a given line to which a parallel through the point $P$ is to be run. First, drop a perpendicular from $P$ to $AB$ by the method described in the preceding article; let $C$ be the point where this perpendicular meets $AB$. Measure $CP$. At any other point $D$ in the line $AB$, erect a perpendicular to the latter line, and measure on this perpendicular a distance $DQ$, equal to $CP$. A line through $P$ and $Q$ will then be the required parallel.

**40. To Determine the Angle Between Two Lines.**—In Fig. 18, $AD$ and $AE$ are two lines on the ground, forming an angle at $A$, the value of which is required. To find this angle, lay off along $AD$ and $AE$ the equal distances $AB$ and $AC$, and measure the distance $BC$. Then the angle $DAE$ is calculated from the relation

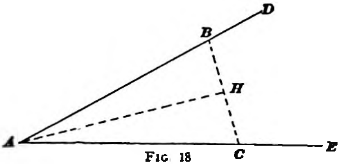


FIG. 18

$$\sin \frac{1}{2} DAE = \frac{\frac{1}{2} BC}{AB}$$

The derivation of this equation is as follows: The triangle $BAC$ being isosceles, the perpendicular $AH$ on $BC$ bisects both the angle $DAE$ and the base $BC$; that is, $BAH = \frac{1}{2} DAE$, and $BH = \frac{1}{2} BC$.

Then in the right triangle $BAH$, sin $BAH=\frac{BH}{AB}$; that is,

$$\sin\frac{1}{2}\,DAE=\frac{\frac{1}{2}BC}{AB}$$

The value of $\frac{1}{2}$ $DAE$ may be found from a table of natural sines; the result multiplied by 2 is the angle $DAE$. For convenience in the calculation, the distances $AB$ and $AC$ should be made 100 feet.

EXAMPLE.—If $AB$ and $AC$, Fig. 18, are each 100 feet long and $BC$ measures 57.6 feet, what is the value of the angle $DAE$?

SOLUTION.—When the values of $AB$ and $BC$ are substituted in the equation, sin $\frac{1}{2}$ $DAE=\frac{\frac{1}{2}BC}{AB}$, the result is

$$\sin\tfrac{1}{2}\,DAE=\frac{\frac{1}{2}\times 57.6}{100}=.28800$$

Then, $\frac{1}{2}$ $DAE=16°\,44'$ (to the nearest minute) and $DAE=2\times 16°\,44'=33°\,28'$. Ans.

**41. To Lay Out an Angle.**—From the line $AB$, Fig. 19, let it be required to lay out an angle $BAC$ less than 90°.

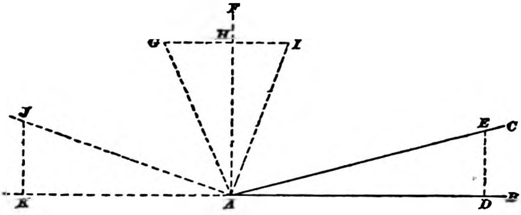

FIG. 19

Lay off any convenient distance $AD$ on $AB$, and at $D$ erect a perpendicular to $AB$. On this perpendicular, lay off $DE$ equal to $AD$ tan $BAC$; and join $A$ with $E$. Then $DAE$ is the required angle.

If the angle is between 90° and 180°, the line $AB$ is prolonged to $K$; then from $AK$, an angle is laid off equal to the difference between 180° and the given angle. For instance,

the angle $BAJ$ is laid off by erecting the perpendicular $KJ$ and taking the distance $KJ$ equal to $AK$ tan $(180° - BAJ)$.

If there are several angles at $A$ not far from 90°, it will be found more economical in the field to erect a perpendicular $AF$ first. If an angle is less than 90°, as $BAI$, lay off $HI$ equal to $AH$ tan $(90° - BAI)$; then $AI$ is a side of the required angle. If an angle is more than 90°, as $BAG$, lay off $HG$ equal to $AH$ tan $(BAG - 90°)$; then $AG$ is a side of the required angle.

For convenience in computing, the distance $AD$, $AK$, or $AH$ should be chosen 100 feet, except when it makes the offset $DE$ or $JK$ too large. In such a case, $AD$, $AK$, or $AH$ may be reduced to 50 feet, or perhaps 20 feet.

EXAMPLE 1.—If the angle $BAG$, Fig. 19, is 114° 45′, and the distance $AH$ is 100 feet, what is the length of the perpendicular $HG$ to be laid off from $AF$ in constructing the angle $BAG$?

SOLUTION.—The angle $FAG$ equals $114° 45' - 90° = 24° 45'$, and $HG = 100 \tan 24° 45' = 46.10$ ft. Ans.

EXAMPLE 2.—What is the length of the perpendicular $KJ$, Fig. 19, to be laid off from $AK$ in order to make the angle $BAJ$ equal to 152° 30′, the distance $AK$ being 100 feet?

SOLUTION.—The angle $KAJ$ equals $180° - 152° 30' = 27° 30'$. Therefore, $KJ = 100 \tan 27° 30' = 52.1$ ft. Ans.

**42. To Prolong a Line Through an Obstacle.**—Suppose that the line $AB$, Fig. 20, is to be prolonged and that an

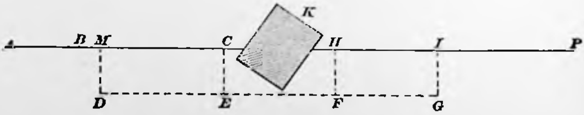


FIG. 20

obstacle $K$, such as a building, makes it impossible to produce the line by a direct sight on poles held at $A$ and $B$. Under such circumstances, the line $AB$ is run to a point $C$ as near the obstacle as possible. At points $C$ and $M$ on the line $AB$, perpendiculars are then erected; the distance $CM$ should be made equal to 30 or 40 feet for convenience in erecting per-

pendiculars by the method explained in Art. **37.** Along these perpendiculars equal distances $MD$ and $CE$ are laid off, of such length that a line through $D$ and $E$ will clear the obstacle; the line through $D$ and $E$ is then parallel to $AC$. Next, the line $DE$ is prolonged and on it are located point $F$, just beyond the obstacle, and point $G$, 30 or 40 feet from $F$. Finally, perpendiculars to $DE$ are erected at $F$ and $G$, and on these perpendiculars, the distances $FH$ and $GI$ are laid off equal to $MD$ and $CE$. The points $H$ and $I$ are then in the prolongation of $AB$; and any point such as $P$ can be located on line with $A$ and $B$ by lining in with poles at $H$ and $I$, as explained in Art. **35.** The distance $CH$ is equal to $EF$, which may be measured if required.

**43. To Determine the Distance Between Two Points, Each of Which is Invisible From the Other.**—In Fig. 21,

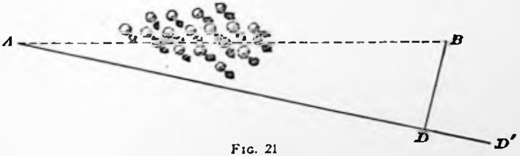


FIG. 21

the points $A$ and $B$ are separated by an obstruction which makes each invisible from the other. A convenient method of determining the distance between these points under such circumstances is as follows: From one of the points, as $A$, a line $AD'$, called a *random line*, is run so that it passes as close as possible to the obstacle. Next, from $B$ a line $BD$ perpendicular to $AD'$ is laid off, and the distances $AD$ and $BD$ are measured. Then in the right triangle $ABD$,

$$\overline{AB}^2 = \overline{AD}^2 + \overline{BD}^2, \text{ or } AB = \sqrt{\overline{AD}^2 + \overline{BD}^2}.$$

As an example, let $AD$ be 206.1 feet and $BD$, 35.1 feet. Then $$AB = \sqrt{206.1^2 + 35.1^2} = 209.1 \text{ feet.}$$

**44. To Determine the Distance Between Two Points When One is Inaccessible.**—Let it be required to determine the distance between points $B$ and $P$, Fig. 22, which are

separated by a river. First, select any point $C$ from which both $B$ and $P$ are visible, and measure $B\,C$. Then take a point $A$ on line with $P$ and $B$, and run line $A\,D'$ parallel to $BC$ by the method described in Art. **39**. On the line $A\,D'$, locate $D$ on line with $P$ and $C$. Finally measure the distances $A\,B$ and $A\,D$. Then,

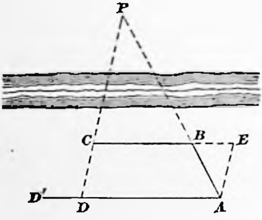

FIG. 22

$$B\,P = \frac{A\,B \times B\,C}{A\,D - B\,C}$$

The derivation of this equation is as follows: Draw $A\,E$ parallel to $D\,P$ and intersecting $C\,B$ produced at $E$. Then, the angles of the triangle $B\,C\,P$ being equal to those of the triangle $A\,B\,E$, these two triangles are similar; therefore, $\frac{B\,P}{A\,B} = \frac{B\,C}{B\,E}$, or

$$B\,P = \frac{A\,B \times B\,C}{B\,E}$$

Since $B\,E = A\,D - B\,C$,

$$B\,P = \frac{A\,B \times B\,C}{A\,D - B\,C}$$

EXAMPLE.—Find the distance $B\,P$, Fig. 22, if $B\,C = 100$ feet, $A\,B = 52.4$ feet, and $A\,D = 124.2$ feet.

SOLUTION.—By the formula,

$$B\,P = \frac{A\,B \times B\,C}{A\,D - B\,C} = \frac{52.4 \times 100}{124.2 - 100} = 216.5 \text{ ft.} \quad \text{Ans.}$$

---

### EXAMPLES FOR PRACTICE

1. If, in Fig. 18, $A\,B$ and $A\,C$ are each 100 feet and $B\,C$ is 64.8 feet, what is the value of the angle $D\,A\,E$? Ans. 37° 48′

2. If the distances $A\,B$ and $A\,C$, Fig. 18, are each 120 feet and the distance $B\,C$ is 96.7 feet, what is the value of the angle $D\,A\,E$? Ans. 47° 32′

3. If the angle $B\,A\,I$, Fig. 19, is to be 56° 30′, and the distance $A\,H$, 80 feet, what should be the length of the perpendicular $H\,I$? Ans. 53.0 ft.

4. If an angle $BAJ$, Fig. 19, is to be constructed equal to 156° 15′, what must be the length of the perpendicular $JK$, when $AK$ is equal to 100 feet? Ans. 44.0 ft.

5. If, in Fig. 21, $AD = 192.5$ feet and $BD = 12.6$ feet, what is the distance from $A$ to $B$? Ans. 192.9 ft.

6. If the distances $CB$, $AB$, and $AD$, Fig. 22, are, respectively, 75, 42, and 103 feet, what is the distance $BP$?

Ans. 112.5 ft.

## CHAIN SURVEY OF A CLOSED FIELD

### FIELD WORK

**45. Preliminary Examination.**—The first step in the survey of a field is to find the marks and monuments at the corners. For this purpose the assistance of the owners of the field and of the neighboring property will be valuable. The tract should then be studied carefully with a view of finding the best method of making the survey.

**46. Measurements.**—In the survey of a field, the lengths of the sides are always required and the locations of important objects are usually necessary. The measurements for locating these objects from a side of the field are made at the same time as the side is being measured, as will be explained later.

If the boundaries are straight lines, the measurements are made by the methods previously described. In the case of an irregular boundary line, such as $GNA$, Fig. 23, which is the edge of a stream, one or more straight lines are run as close as possible to the boundary. From these auxiliary lines, perpendicular measurements, called *offsets*, are taken to those points on the boundary where any considerable change in direction occurs.

Thus, in Fig. 23, the straight survey line $GA$ is run close to the shore line $GNA$. The distances $GH$, $GJ$, and $GL$ and the offsets $HI$, $JK$, and $LM$ are measured and recorded in the notebook while the line $GA$ is being run. It will be seen that all distances along a main or auxiliary line for taking

offsets are measured from the beginning of the line. For instance, instead of taking the distances *GH*, *HJ*, and *JL*, the distances *GH*, *GJ*, and *GL* are recorded. The parts *GI*, *IK*, *KM*, and *MA* of the boundary being nearly straight, the portion of the field between the straight line *GA* and the irregular line *GNA* is divided so as to form approximately the right triangles *GHI* and *LMA*, and the trapezoids *HIKJ* and *JKML*.

It is often difficult to measure directly along a line, as when a boundary is marked by a fence. In such cases, an

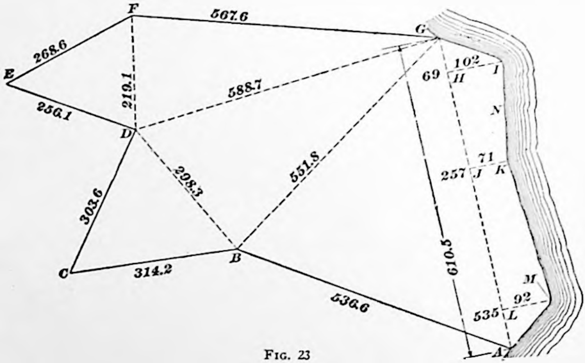


Fig. 23

offset of 3 or 4 feet is measured at each end of the line and the distance between the ends of these offsets is taken and recorded as the length of the line. The directions of such short offsets can be estimated by eye with sufficient accuracy.

**47. Dividing a Field Into Triangles.**—In order to make a plot of a field like that shown in Fig. 23 and to calculate its area, the field is divided into triangles by means of diagonals, which are measured on the ground. The surveyor should use his own judgment as to the best and most convenient diagonals to measure. He should avoid using diagonals that make triangles with very acute or very obtuse angles. Thus in

Fig. 23. $DG$ is a better diagonal to use than $BF$. Extra diagonals should be measured occasionally to check the work.

**48. Tie-Lines.**—Obstacles often make it impossible to measure directly the diagonals of a field. In such cases, their lengths may be determined by the methods described in Arts. 42, 43, and 44, or by the process illustrated in Fig. 24, which represents a field $ABCDE$ in which the diagonals $BD$ and $BE$ cross a pond, and cannot, therefore, be measured directly. To find $BE$, the side $BA$ is first prolonged to some point $F$ and the side $EA$ is prolonged to $G$, the distance $AG$ being calculated from the proportion $AB:AF = AE:AG$;

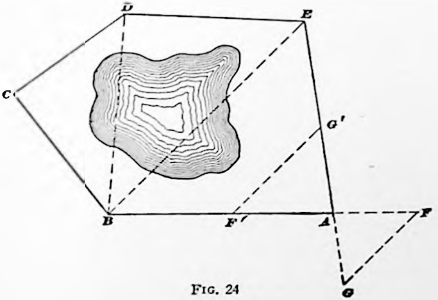


FIG. 24

thus, the triangle $FAG$ is made similar to $BAE$. Then, the distance $FG$ is measured and $BE$ is computed from the relation $AF:AB = FG:BE$. For convenience, the distances $AF$ and $AG$ should be made some simple fractional part of $AB$ and $AE$, such as $\frac{1}{2}$, $\frac{1}{3}$, or $\frac{1}{4}$. Then, $BE$ will be 2, 3, or 4 times $FG$.

For example, suppose that the sides $BA$ and $EA$ are 320 feet and 304 feet, respectively, and that $AF$, in the prolongation of $BA$, is made equal to $\frac{1}{4}$ $AB$, or $\frac{1}{4}\times 320 = 80$ feet. Then, the distance $AG$, in the prolongation of $EA$, must be laid off equal to $\frac{1}{4}$ $AE$, or $\frac{1}{4}\times 304 = 76$ feet. In this case, $FG$ will be $\frac{1}{4}$ $BE$, or $BE$ will be equal to 4 $FG$; if the length of $FG$ is found to be 107 feet, the length of $BE$ equals $4\times 107 = 428$ feet.

Instead of constructing a triangle such as $AFG$ by producing the lines $AB$ and $AE$, a triangle $AF'G'$ may be laid off inside the tract. If $AF'$ and $AG'$ are made proportional to $AB$ and $AE$, respectively, the line $F'G'$ will be in the same proportion to $EB$ as $AF'$ is to $AB$, or as $AG'$ is to $AE$.

Such lines as $FG$ and $F'G'$ are called *tie-lines* because they tie the sides together. The length of the diagonal $BD$ can be determined by means of tie-lines in a similar manner.

**49. Location of Objects.**— It is often desirable to locate important objects, such as buildings, roads, fences, etc., with

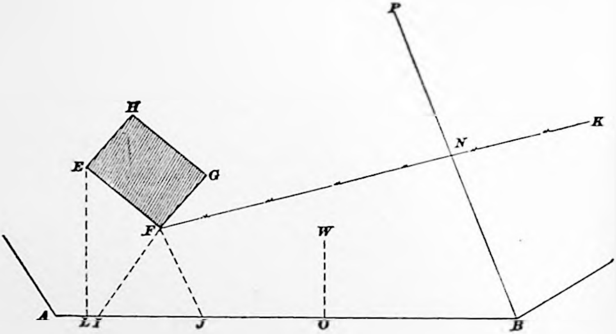


FIG. 25

reference to the main lines of a survey. Points very near the main line are most easily located by measurements along the line and perpendicular distances, or offsets, from the line. The points along the main line from which the offsets are to be measured can be determined by eye while the line is being run, and can be marked on the ground temporarily. The distances from the preceding corner and the lengths of the offsets should be measured and recorded. In Fig. 25, the point $W$ may be located from $AB$ by measuring the distance $AO$ and the perpendicular offset $OW$.

When a point is so far from the main line that the direction of the perpendicular offset cannot be estimated accurately

by eye, the method by a perpendicular offset is not convenient. In such a case, the point is located best by measuring the distances from two suitable points on the main line. For instance, in Fig. 25, the house corner $F$ can be located by measuring the distances $IF$ and $JF$. The directions of the sides of the house can be conveniently determined by taking the point $I$ in the prolongation of $FG$.

A rectangular object can be referenced by locating two corners and measuring the dimensions of the object. For the building $EFGH$ in Fig. 25 the point $F$ can be located from $AB$, $E$ can be located by distances from $F$ and $L$, and $G$ and $H$ can be determined from $E$ and $F$ by measuring the dimensions of the building.

The direction of a straight line is determined by locating two points on it. For instance, a straight fence or road is located when two points on it are known. Thus, the fence $FK$, Fig. 25, may be referenced by locating the points $F$ and $N$; $F$ may be located from $AB$, as previously explained, and $N$ may be located when the line $BP$ is being measured.

Corners of a field and other important stations should be *witnessed* or *referenced* by measurements to nearby objects, such as trees and corners of buildings, which are permanent and easily found. The purpose is to aid the surveyor to rerun his line, if necessary. He can thus either find or replace all important points.

**50. Keeping Notes.**—The notes, or record, of a chain survey are usually kept in an ordinary field book, commonly called a *transit book*, which fits in the pocket. The left-hand page of the notebook is ruled in six columns, as shown in Fig. 26. In one column are written the letters or station numbers by which the corners or important points are designated. Horizontally opposite each letter or number, in the next column, is recorded the distance of the point from the corner immediately preceding it. As only two columns of this page are needed, the fourth and fifth columns are used for this purpose, so as to bring them nearer the right-hand page, and to leave plenty of space to the left for remarks.

The right-hand page is ruled in blue with a vertical red line at the center. This page is used for sketches and remarks. The survey line is commonly represented by the red center line. In case more room is needed for sketching, the line being run may be drawn on one side of the center line of the page and parallel to it. On this page are noted also the date and location of the survey, the names and positions of the different members of the party, and any other remarks that the surveyor may deem necessary and useful. In sketching, it is better to face in the direction in which the line is being measured, and to represent the line as running from the bottom

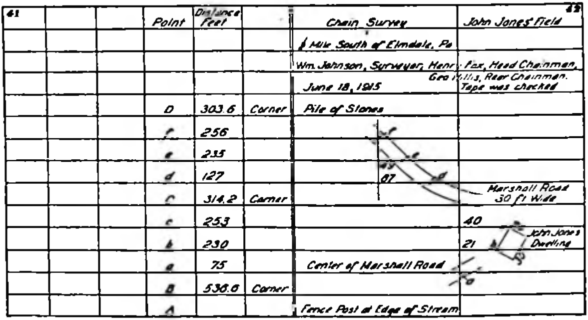


Fig. 26

toward the top in the notebook. For this reason, nearly all surveying notes read upwards from the bottom of the page.

The notes given in Fig. 26 are for part of the field the boundaries of which are represented in Fig. 23. The survey is supposed to have begun at *A*, from which the line was run to *B*, then to *C*, etc. The number 536.6 opposite *B*, denotes the distance from *A* to *B*; the distance from *B* to *C*, 314.2, is recorded opposite *C*; and so on for the other corners. The numbers between corners, as those between *B* and *C*, are the distances from the corner immediately preceding to the points directly opposite in the sketch. Thus, 230 and 253 are the distances from *B* to the points on *BC* where offsets were

taken to the corners *b* and *c* of the dwelling indicated in the sketch. In locating a road, measurements are usually taken to the center line of the road instead of to one edge; thus, the distances 87 and 49 are to the center of Marshall road. On the right-hand page the survey line *CD* is represented by the vertical line to the left of, and parallel to, the line at the center of the page.

Notes should be full and plain, and should be kept as neatly as possible. The surveyor should keep his notes in such a manner that they can be readily understood, not only by himself, but by any one having a knowledge of surveying. This is especially necessary when the notes are not to be plotted by the same person who takes them. The pages of the notebook should be numbered. All corners and important stations should be fully described. If these points are mentioned in another survey, reference should be had to that survey.

---

## OFFICE WORK

**51. Plotting the Notes.**—As a map shows the relative positions of points on the ground, the length of any line on the map is proportional to the length of the corresponding line on the ground. The distance on the ground represented by a unit distance on the map is called the *scale of the map*. For example, if 1 inch on the map represents 100 feet on the ground, the scale is said to be *100 feet to the inch*, or *1 inch = 100 feet;* if 1 inch on the map represents 500 feet on the ground, the scale is *1 inch = 500 feet*, etc. Since 100 feet equals $100 \times 12$, or 1,200, inches, a scale of 1 inch = 100 feet is the same as a scale of 1 inch = 1,200 inches; such a scale is, therefore, often called *1 to 1,200.* Similarly, a scale of 1 inch = 500 feet, or 6,000 inches, is sometimes called *1 to 6,000.*

For convenience in plotting distances to a given scale, a graduated rule, known as an *engineer's scale*, is commonly used by surveyors. These scales are usually made of boxwood, are 12 inches long, and are triangular in shape, as shown in Fig. 27. Flat scales, 6 inches long, are also made for convenience in carrying in the pocket. The scale shown in Fig. 27 has six

systems of graduations, one on each side of each edge; it is, therefore, a combination of six scales. Each scale is so divided that the number of divisions in 1 inch is a multiple of ten, this number being indicated by large figures in the center of the scale; thus the numbers 10 and 50, Fig. 27, indicate that the scales are divided to tenths and fiftieths of an inch. Every tenth graduation mark is long, and the number of long graduations from the zero of the scale is shown by the figures. Hence, the number 32 on the 50-scale indicates a distance of $32 \times 10$, or 320, divisions from the zero of the scale; the actual distance in inches is unimportant.

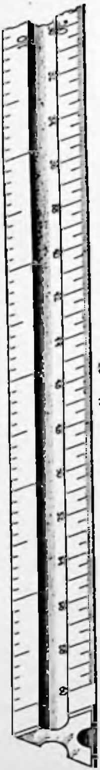

Fig. 27

**52.** The best scale to use in plotting a field is determined by the size of the field and the purpose of the map. If a scale of 1 inch = 10 feet, 1 inch = 100 feet, or 1 inch = 1,000 feet is chosen, the 10-scale will be most convenient, since then each small division will represent 1 foot, 10 feet, or 100 feet, respectively. For a scale of 1 inch = 40 feet, 1 inch = 400 feet, etc., the 40-scale is best; then each small division is 1 foot, 10 feet, etc. Readings to smaller values can readily be made by estimating parts of a division.

Suppose that a scale of 1 inch = 200 feet is used, and it is desired to lay off a line 283 feet long. Evidently, the 20-scale is best; then each small division represents 10 feet, and each long graduation indicates 100 feet. With the zero of the scale, $A$ in Fig. 28 ($a$), opposite the beginning of the line, the graduation opposite the number 2 on the scale indicates 200 feet. The point $B$ at the end of the measurement is then located 8.3 divisions beyond that long graduation, the decimal part of a division being estimated by eye. Suppose again that a line drawn to a scale of 1 inch = 600 feet is measured

with the 60-scale; in this case, also, each division represents 10 feet, and each long graduation indicates 100 feet. With the zero of the scale, *A* in Fig. 28 (*b*), at the beginning of the line, the distance to the end of the line, at *B*, is found as follows: The numbered graduation just before *B* is 12 and the unnumbered long graduation between 12 and *B* is 13, which indicates 1,300 feet. Then, since the point *B* is exactly opposite the fourth division past the long graduation, the distance from that mark to *B* is 40 feet. Hence, the length of the line *AB* is 1,300+40 or 1,340 feet.

**53.** A plot of the field represented in Fig. 23 would be constructed in the following manner: Any line of the plot may be drawn first, but it should be so located that the map will come within the limits of the paper and be approximately in the center. The proper position can be determined by inspection of the sketch and the notes. In the present case it will be assumed that the line *FG* is drawn first; its length is made equal to 567.6 feet to the scale selected for the map. Then from *F* and *G* as centers, and with radii representing 219.1 and 588.7 feet, respectively, two arcs are described; the point of intersection locates point *D* on the map. Point *E* can be located by describing arcs from *F* and *D*; point *B* can be determined from *G* and *D*; and the other corners can be located in a similar manner from points previously found. Then the boundaries of the field are obtained by drawing straight lines between the points, or corners. The plot should be checked by the extra measurements made in the field and not used in locating the corners on the map.

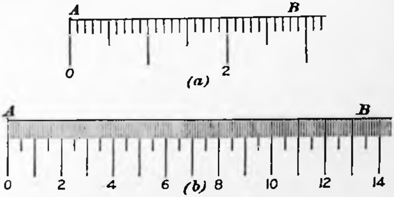


FIG. 28

After *GA* has been plotted, the points on the irregular boundary *GNA* are located by laying off distances *GH*,

$GJ$, and $GL$, and erecting perpendiculars $HI$, $JK$, and $LM$, having the proper lengths. By drawing a freehand line through points $G$, $I$, $K$, $M$, and $A$, and making the parts between these points nearly straight, the irregular boundary is determined.

**54. Calculating the Area.**—The area of a field is obtained by calculating the areas of the triangles and trapezoids into which the field has been divided and then taking the sum of these partial areas.

When the lengths of the three sides of a triangle are known, its area can be found by the formula,

$$S = \sqrt{s(s-a)\,(s-b)\,(s-c)} \qquad (1)$$

in which $S$ = area in square feet;
$a$, $b$, and $c$ = lengths of sides, in feet;

and $$s = \frac{a+b+c}{2}$$

When the parallel bases of a trapezoid and the perpendicular distance between them are known, the area can be calculated by the formula

$$T = d\left(\frac{m+n}{2}\right) \qquad (2)$$

in which $T$ = area in square feet;
$m$ and $n$ = parallel bases, in feet;
$d$ = perpendicular distance between bases, in feet.

The area in square feet can be changed to acres by dividing by 43,560, since there are 43,560 square feet in an acre.

EXAMPLE 1.—Find the area of the triangle $BCD$ in Fig. 23.

SOLUTION.—Here, the three sides are $a = 303.6$ ft., $b = 298.3$ ft., and $c = 314.2$ ft. Then, $s = \frac{a+b+c}{2} = \frac{916.1}{2} = 458.1$, $s-a = 154.5$, $s-b = 159.8$, and $s-c = 143.9$. By formula **1**, the area of the triangle equals

$$S = \sqrt{458.1 \times 154.5 \times 159.8 \times 143.9} = 40{,}340 \text{ sq. ft.} \quad \text{Ans.}$$

NOTE.—In computing an area from measurements which are not very accurate, four significant figures are sufficient.

EXAMPLE 2.—Find the area of the trapezoid $HIKJ$ in Fig. 23.

SOLUTION.—Here, $M = 102$ ft., $n = 71$ ft., and $d = 257 - 69 = 188$ ft., Then, by formula 2,

$$T = 188\left(\frac{102+71}{2}\right) = 16{,}262 \text{ sq. ft.} \quad \text{Ans.}$$

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## EXAMPLES FOR PRACTICE

1. Find the area of the triangle $DEF$ in Fig. 23. Ans. 26,070 sq. ft.

2. Find the area of the trapezoid $JKML$ in Fig. 23. Ans. 22,657 sq. ft.

# LEVELING

Serial 3068-2 Edition 1

## DIRECT LEVELING

### INTRODUCTION

#### DEFINITIONS

**1. Vertical Lines.**—A line in the direction of a plumb-line is vertical. For ordinary purposes it is convenient to assume that the earth is a true sphere with a smooth surface, and that a plumb-line held at any point on its surface is always directed toward the center of the sphere. Thus, if *O*, Fig. 1, is the center of the earth and *A* and *B* are two points on the earth's surface, then a vertical line has the direction *OA* at *A* and *OB* at *B*.

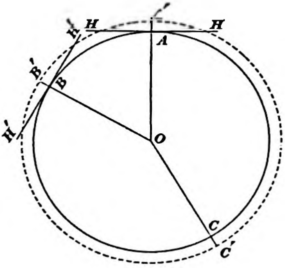

Fig. 1

**2. Level Surfaces.**—A surface, which at each point is at right angles to the direction of a plumb-line at the point, is a level surface; a line in such a surface is a level line. Every level surface is, therefore, assumed to be a part of a sphere having its center at the center of the earth. Thus, in Fig. 1, the circle *ABC*, representing the earth's surface, and the circle *A'B'C'*, representing a level

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surface, have the same center $O$. It should be kept in mind that a level surface is not a plane.

**3. Elevations.**—The vertical distance of any point above or below some level surface, adopted as a base for reference, is the elevation of the point with respect to the base. The base, or reference, surface is generally called a *datum*.

**4. Leveling** is the operation of determining the elevations of a series of points.

**5. Horizontal Surfaces.**—A horizontal surface at any point is a plane that is tangent to a level surface at that point; a line in such a surface is a horizontal line. A horizontal surface is perpendicular to a plumb-line at the point where it is tangent to a level surface. For example, in Fig. 1, $HH$ represents a horizontal surface at the point $A$, and $H'H'$ represents a horizontal surface at $B$. Since the radius of curvature of the earth's surface is very great, a horizontal surface and a level surface will very nearly coincide for a considerable distance in every direction from the point of tangency. Hence, any reasonably short horizontal line may for ordinary purposes be considered to be a level line, and is commonly so considered. There are cases, however, in which the curvature of the earth's surface cannot be neglected.

**6. Sea Level.**—A datum may be an imaginary surface or the actual surface, commonly called sea level, which is the surface of the sea exactly midway between high and low tides. Sea level is the datum most generally used because it is the same at all points on the earth's surface and, therefore, furnishes a universal standard. The elevation of a point with respect to sea level is commonly termed its *altitude*.

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## METHODS OF LEVELING

**7.** Elevations are determined by three general methods, which differ with regard to the principles involved and the instruments and processes employed. They are: (1) *Direct leveling*, sometimes called *spirit leveling*, and also designated as

*gravity leveling;* (2) *trigonometric leveling,* also known as *indirect leveling;* and (3) *barometric leveling.*

8. **Direct leveling** is the method of determining the elevations of points by measuring their vertical distances above or below a level line or a series of level lines. The device universally used for determining when a line is level is the *spirit level,* described later, from which fact this method of leveling is often called *spirit leveling.* The name *gravity leveling* is also sometimes employed because a level line is always at right angles to the direction of gravity. Direct leveling is the method used when a high degree of accuracy is required, and it is also employed when conditions make it more convenient than the other methods.

9. **Trigonometric leveling** is a method of determining the difference in elevation between two points by measuring the horizontal or the inclined distance between them and determining the angle between a horizontal line and the inclined line that joins the given points. The required difference in elevation is then one leg of a right triangle in which one acute angle and the other leg or the hypotenuse are known. It is a convenient method to use when the elevations of principal stations only are required. If carefully done, it is almost as accurate as direct leveling. Trigonometric leveling requires the measurement of angles, and is treated in the Section on *Transit Surveying.*

10. **Barometric leveling** is a method of determining the approximate difference in elevation between two points by measuring the difference between the atmospheric pressures at the points.

## THE ENGINEERS' LEVEL

### TYPES OF LEVELS

**11.** The instrument most extensively used in leveling is the *engineers' level*. It consists essentially of a telescope, having a very accurate spirit level attached longitudinally. The telescope is supported at the ends of a straight bar, which is firmly secured at the center to a perpendicular axis on which it revolves. The whole is supported on a *tripod*.

There are two general classes of engineers' levels: the *wye level*, also written Y *level*, in which the telescope rests in Y-shaped supports from which it can be removed; and the *dumpy level*, in which the telescope is rigidly attached to the bar supporting it.

The wye level was formerly preferred by American engineers for ordinary work because of the ease with which it can be adjusted, but the dumpy level is steadily gaining in favor. For very accurate work a form of the dumpy level, known as the *precise level*, has been found superior to the wye type because it has few movable parts and does not get out of adjustment easily.

### THE WYE LEVEL

**12. Description.**—An engineers' wye level is shown in Fig. 2. The *telescope a b*, having attached to it an accurate and sensitive *spirit level c d*, rests in the Y-shaped supports *e* and *f*, in which it is held firmly by semicircular clasps, commonly called *clips*. The clips are hinged at one end, and, passing over the telescope, are held at the other end by small pins; the pins can be removed, and in order that they should not be lost, they are fastened to the supports by short cords. When the clips are open, the telescope can be turned in its supports so that the spirit level is no longer directly beneath the telescope. When the instrument is in use, however, the telescope must be prevented from turning. For this purpose, various devices are used; sometimes a small projection on

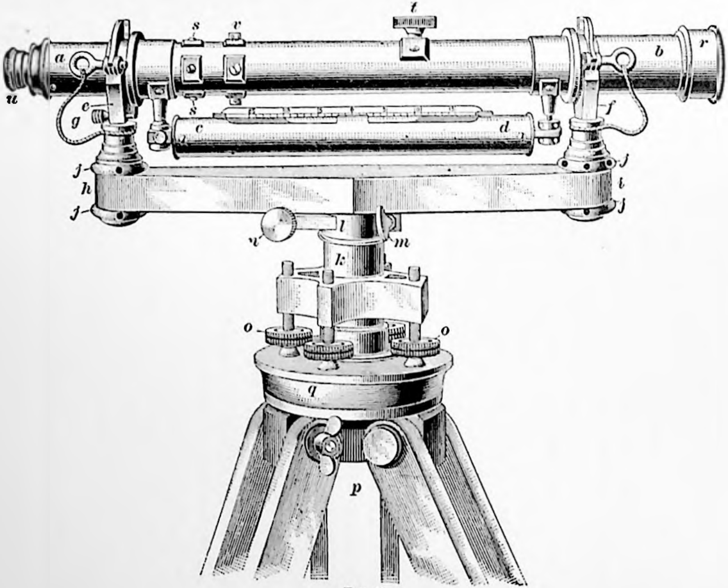


FIG. 2

the telescope bears against the stop-piece *g* but the best method is to have pins on the clips fit in holes in the top of the telescope.

The Y-shaped supports, or *wyes*, are distinguishing features of this form of level and from them the instrument derives the name *wye level*. The lower ends of the wyes pass through the ends of the horizontal bar *h i*, called the *level bar*, and are adjustable vertically by means of the capstan-pattern nuts *j*, which bear against the upper and lower surfaces of the bar. The bar *h i* is attached rigidly to a *center*, or *spindle*, which turns in the socket *k*. A collar *l*, which is connected to the level bar by means of a projection from the bar, revolves on the socket. When the clamp screw *m*, which is part of the collar, is loose, the telescope can be rotated in a horizontal plane. The instrument can be secured against rotation by tightening the screw *m*, which then holds the collar fixed on the socket. After the clamp *m* has been tightened, the telescope can be revolved slowly through a small angle by means of a screw *n*, known as a *tangent screw*. The projection from the level bar fits between the point of the tangent screw and a resisting spring. When the screw is tightened, its point pushes the projection, the spring is compressed, and the telescope rotates. When the tangent screw is loosened, the spring forces the projection from the level bar back against the point of the screw, and the telescope rotates in the opposite direction. The inclination of the socket *k* is controlled by the *leveling screws o*, which are four in number on some instruments and only three on others. The instrument is supported on a *tripod*, which consists of three legs shod with steel and connected by hinge joints to a metal *tripod head*. The upper part of the tripod is shown at *p* in Fig. 2. The tripod head is threaded in order that the plate *q* of the level can be screwed on.

**13. Telescope.**—A longitudinal section through the telescope *a b* is shown in Fig. 3. The essential parts are the *objective w*, the *eyepiece x x*, and the *cross-hairs*, or *cross-wires*, *y*.

The objective consists of a combination of *lenses* (pieces of glass with curved surfaces). When rays of light from the

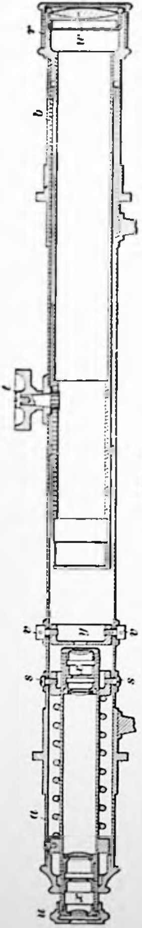

FIG. 3

object sighted at strike the objective, they are deflected to form a figure similar to the object, but inverted. This figure, called the *image*, is located in a definite plane. The cross-wires, described in the following article, should be placed in the telescope as near as possible to the image. Then, when the image and the cross-wires are viewed through the eyepiece, which is another combination of lenses that enlarges the image and the cross-wires, both will be seen clearly at the same time. Some eyepieces show the object upside down; others invert the image so that the object is seen in its natural position. Telescopes with eyepieces of the former class are called *inverting telescopes*, and those having eyepieces of the latter class are known as *erecting telescopes*. Although the eyepiece of an inverting telescope has fewer lenses and the lines of the object are, therefore, more clearly defined, most instruments have erecting telescopes.

The objective is shown covered by a metal cap *r*, Figs. 2 and 3, which protects it when the instrument is not in use. When a sight is taken through the telescope, this cap is removed and a thin metal tube, called a *sunshade*, is used to keep the glare of the sun from striking the objective and making the view of the object indistinct. A device for covering the eyepiece is also supplied.

The small screws at *s* are inserted to center the eyepiece in the telescope; they are necessary only with an erecting telescope, in which the eyepiece is very long.

**14.** The *cross-hairs*, or *cross-wires*, are two very fine platinum wires fastened at right angles to each other in a substantial brass ring or diaphragm. This ring is held in position in the telescope by four capstan-headed screws $v$, Figs. 2 and 3, that pass through holes in the telescope and screw into the ring, as shown in Fig. 4. The holes in the telescope are slightly larger than the screws so that the ring can be rotated in the telescope through a small angle. The ring can also be moved vertically or horizontally by turning the proper capstan screws. Formerly, the cross-hairs were made of clean spider web, but platinum wire has been found more satisfactory.

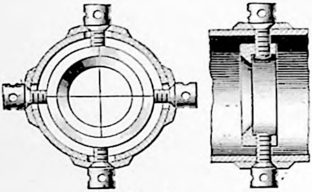

FIG. 4

**15. Focusing.**—As the distance from the telescope to the observed object varies, the location of the image changes. In order to bring the image in the plane of the cross-wires for any sight, the position of the objective in the telescope can be adjusted by the milled wheel $t$, Figs. 2 and 3, which is placed on the top of the telescope in this case, but is sometimes on the right-hand side. Changing the objective may make it necessary also to adjust the location of the eyepiece to obtain a distinct view of the object; this can be done by rotating the milled ring $u$. The operation of adjusting the eyepiece and the objective in order to see the cross-wires and the object distinctly at the same time is called focusing.

The objective is focused when the object sighted at appears clear and distinctly outlined. To focus the eyepiece, the telescope is pointed toward an open space and the eyepiece is moved by turning the ring $u$ until the cross-wires appear sharp and distinct. When both the objective and the eyepiece are focused, the cross-wires will show no movement with respect to the observed object, no matter how the position of the eye may be changed.

**16. Line of Sight and Line of Collimation.**—A line of sight is a line joining any point with the eye of the observer. When a point is observed through a telescope, the line of sight is indicated by the line through the center of the objective and the intersection of the cross-wires. This line is sometimes called the *line of collimation*.

**17. Spirit Level.**—The spirit level *c d*, Fig. 2, is the part of the instrument on which accuracy chiefly depends. It consists of a sealed glass tube, curved slightly to correspond to the short upper arc of a large vertical circle, and so nearly filled with alcohol, or a mixture of alcohol and ether, as to leave only a small bubble of air. This tube, commonly called the *level tube*, is fastened securely, but not rigidly, in a metal case having a long, narrow longitudinal opening in its top through which the glass tube and air bubble can be seen. Each end of the metal case is attached to a stud projecting from the under side of the telescope tube, one end being adjustable vertically by means of capstan-pattern nuts and the other end adjustable laterally by means of capstan-headed screws, as shown in Fig. 2.

A line tangent to the upper surface of the level tube at its center is called the *axis of the bubble tube* or *axis of the level tube*. When the bubble is in the center of the tube, the axis of the bubble tube is horizontal. In order to show the position of the bubble with reference to the center of the tube, a graduated scale is marked either on the tube or on a small strip of silvered brass attached to the upper side of the tube. In Fig. 5 is shown the top view of the glass tube of a spirit level in which the graduations are marked on the tube and the bubble is represented as truly centered. In Fig. 2, the scale is marked on a metal strip; the bubble in this case is also centered.

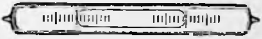

FIG. 5

**18. Axis of Revolution.**—The level has but one axis of revolution, which is the vertical axis of the instrument. In the use of the level, the only essential requisite for accurate work is that the line of sight shall be truly horizontal, and this

condition should be indicated by the bubble's being in the center of the level tube. In other words, the line of sight should be parallel to the axis of the level tube. If the instrument is adjusted properly and is leveled up as described in the next article, the line of sight revolves on the vertical axis in a horizontal plane.

**19. Setting Up.**—The first step in making a level ready for use is to screw the instrument securely on the tripod head and to plant the tripod legs firmly in the ground in such positions that the plate *q*, Fig. 2, is nearly horizontal. If the instrument has four leveling screws, the telescope is rotated until it is over one pair of opposite screws and the bubble is brought to the center of the level tube by turning only these two screws. The screws should be held between the thumb and forefinger of each hand, and they should be turned in opposite directions; that is, the thumbs should move either toward each other or away from each other. Both screws should be turned at the same time and at about the same rate.

After the bubble has been brought to the center of the level tube for this position of the telescope, the telescope is turned on the vertical axis through an angle of 90° so that it is over the other pair of leveling screws. The bubble is then brought to the center of the tube by means of these two screws. The telescope is turned back to its first position, care being taken not to have it reversed end for end, to see if the bubble remains in the center. If it does, the instrument is leveled; if it does not, it is brought to the center over each pair of leveling screws alternately, until it stays in the center for both positions of the telescope.

If there are three leveling screws, the telescope is first placed parallel to the line through any two of them, and the bubble is brought to the center of the tube by means of these two screws. Then the telescope is revolved until it is over the third screw and the bubble is brought to the center by means of this screw alone. If the bubble remains in the center when the telescope is brought back to its first position, the instrument is leveled. Otherwise, the operations must be

repeated until the bubble remains in the center for both positions of the telescope.

The expression *setting up the level* will be considered to include making the line of sight horizontal as well as merely placing the tripod legs in position, since the instrument must always be leveled before it can be used for determining elevations.

**20. Care of Level.**—The level should not be exposed to the sun, to rapid changes of temperature, to unequal temperatures on its different parts, to dust, or to rain when such exposure can be avoided. Changes of temperature disturb the adjustments, dust is injurious to the bearings and the lenses, while moisture obscures the lenses and is otherwise injurious to the instrument. When it is impossible to avoid working in the rain, wipe the lenses frequently and carefully with a soft linen cloth and, when the instrument is not in use, cover the eyepiece and put the cap on the objective. After returning to the office or camp, wipe the entire instrument very carefully and thoroughly, finishing with a piece of dry chamois skin; then, leave it in a moderately warm, dry place, so that every particle of moisture will be removed.

When a level is carried on its tripod in open country, the spindle should always be clamped slightly to prevent the wearing of the centers by swinging, and the instrument should be carried with the object end of the telescope down. In a wooded country where underbrush is dense, the level should be carried with the spindle unclamped, so that the telescope will turn freely on the spindle and yield readily to any pressure. A blow that would inflict no injury upon an unclamped instrument might seriously damage one clamped rigidly.

Care should be exercised not to use unnecessary force in screwing the instrument on the tripod, in tightening the clamp screw, or in turning the leveling screws. If the leveling screws bind, two adjacent screws should be loosened slightly. When the instrument is in use, all screws should bear firmly, but excess pressure is likely to cause damage.

### ADJUSTMENTS OF WYE LEVEL

**21.** There are three important adjustments of the **Y** level, as follows:

1. To make the line of sight parallel to the line through the lowest points of the collars on the telescope, which rest in the wyes.

2. To make the axis of the level tube parallel to the line through the bases of the collars, and, consequently, parallel to the line of sight.

3. To make the axis of the level tube perpendicular to the vertical axis of the instrument, so that, when the instrument is leveled up, the bubble will remain centered while the telescope is revolved horizontally.

The first two adjustments are sufficient as far as accuracy of the instrument is concerned. The third adjustment affects only the rapidity with which the work can be performed. When this adjustment is made and the instrument is leveled up carefully, the bubble will remain centered in whatever direction the telescope is turned. But if this adjustment is poor, the bubble will have to be recentered by the leveling screws every time the telescope is pointed in a new direction.

There is also a preliminary adjustment by which the horizontal cross-wire is made truly horizontal when the instrument is leveled. This adjustment is not necessary, but it permits the use of any part of the horizontal wire instead of only the point of intersection of the cross-wires.

**22. Preliminary Adjustment.**—If the cross-wires are in their original positions in the ring, it may be assumed that when the vertical wire is truly vertical the horizontal wire is exactly horizontal. It is then convenient to make the preliminary adjustment by bringing the vertical wire to a vertical position. Also, a truly vertical wire is helpful in direct leveling. In making the adjustment, the level is first set up at a suitable place and the telescope is leveled over both pairs of opposite leveling screws. The direction of the vertical wire is then compared with the line of a plumb-bob suspended at

some distance from the instrument or with the vertical edge of a building. If the wire deviates from the vertical line, two adjacent capstan-headed screws, $v$ in Fig. 2, are loosened and the cross-wire ring is rotated carefully by pressing against the heads of the screws or by tapping them lightly, until the wire coincides with the vertical line. If the telescope is inverting, the wires are seen in their true positions, and the ring should be rotated in the direction that seems necessary. But, if the telescope is erecting, the wires appear inverted, and the ring should be turned in the direction opposite to that in which the image of the wires should be rotated. Thus, if $ab$, Fig. 6, indicates the position of the plumb-bob and $cd$ represents the vertical wire, the ring should be rotated in the direction of the upper arrow for an inverting telescope and in the direction of the lower arrow for an erecting telescope. When the cross-wires are in the proper positions, the loosened screws should be tightened. If it is suspected that the cross-wires are not exactly at right angles, as when one wire has been replaced, it is preferable to adjust the horizontal wire by comparing its direction with a truly horizontal line.

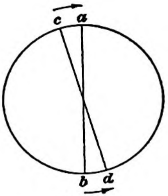

FIG. 6

**23. First Adjustment, or Adjustment of Line of Sight.** To make the first adjustment, plant the tripod firmly; but it is not necessary to level the telescope. Sight toward some distant vertical surface, such as a fence or building, clamp the instrument by means of the screw $m$, Fig. 2, and mark a point that coincides with the intersection of the cross-wires. Remove the pins. loosen the clips that hold the telescope in the wyes, and revolve the telescope in the wyes through one-half a revolution, that is, until the bottom side is up and the bubble tube is above the telescope. If the intersection of the cross-wires is still on the marked point, the line of sight is parallel to the line through the bottoms of the collars. If the intersection of the cross-wires is no longer on the point, mark the new position on the surface.

Suppose that the initial positions of the cross-wires are shown by the full lines in Fig. 7 and the point *a* at their intersection is marked. Suppose further that, after revolution of the telescope, the positions of the cross-wires are shown by the broken lines of short dashes, and the point of intersection *b* is marked on the surface near *a*. Then draw a straight line from *a* to *b* and mark its middle point *c*, which is the proper position of the point of intersection.

To bring the point of intersection to *c*, first move one of the cross-wires, say the horizontal, to *c* by means of the upper and lower capstan screws, and then set the vertical wire on *c* by means of the capstan screws on the sides. Both wires should not be moved at once since the cross-wire ring is liable to be rotated, and the preliminary adjustment to be spoiled. In moving a cross-wire, one capstan screw is always loosened first and then the opposite screw is tightened a corresponding amount. It is advisable to move the wire in short steps rather than large amounts at a time.

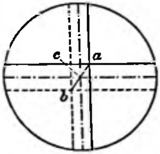

Fig. 7

In an inverting telescope, a wire is adjusted by loosening the screw away from which the image of the wire is to be moved and tightening the opposite screw. Thus, since the image of the horizontal wire in Fig. 7 should be moved upwards from *b* to *c*, the lower screw is loosened and the upper screw tightened; since the image of the vertical wire is to be moved to the right, the left-hand screw is loosened and the right-hand screw tightened.

To adjust a wire in an erecting telescope, loosen the screw toward which the image of the wire must be moved and tighten the opposite screw. In the case shown in Fig. 7, the upper screw is loosened and the lower one tightened for the horizontal wire, and the right screw loosened and the left tightened for the vertical wire.

**24. Second Adjustment.**—This adjustment is made in two parts, the second being somewhat dependent on the first. The first part is for adjusting the level tube laterally so that

it can be adjusted accurately in a vertical direction. The work is done as follows:

Set up the instrument, clamp the spindle with the telescope directly over one pair of opposite leveling screws, and bring the bubble to the center of the tube by means of these two screws. Revolve the telescope in its wyes until the bubble tube is no longer directly under the telescope. If the bubble remains in the center of the tube, no lateral adjustment is necessary. But if the bubble leaves the center of the tube, bring it very nearly back to the center by means of the capstan-headed adjusting screws on the sides at one end of the level tube. Then revolve the telescope in its wyes back to its initial position, bring the bubble to the center of the level tube by means of the leveling screws, and test again to see whether it stays in the center when the telescope is revolved in the wyes. Repeat these operations as often as required.

Having adjusted the level tube laterally, bring the bubble exactly to the center of the tube by means of the leveling screws. Then lift the telescope out of the wyes and replace it with its ends reversed. In handling the telescope, take care not to disturb the position of the wyes. If the bubble is again in the center of the tube, the level tube is properly adjusted. However, if the bubble is not in the center, bring it half way back to the center by means of the capstan-pattern nuts at one end of the level tube. If the bubble is nearer the end of the tube with the nuts, lower that end by first loosening the lower nut and then tightening the upper nut. If the bubble is nearer the fixed end of the tube, raise the end with the nuts by first loosening the upper nut and then tightening the lower nut.

Again bring the bubble exactly to the center of the tube by means of the leveling screws and reverse the telescope in its wyes to see whether the bubble stays in the center. If it does not, repeat the operations until it does.

**25. Third Adjustment.**—To make the axis of the level tube perpendicular to the vertical axis of the instrument, level up the telescope first over one pair of opposite leveling screws and then over the other pair. From this second posi-

dumpy level, the telescope $a\ b$ is attached rigidly to the level bar $c\ d$. Second, the level tube $e\ f$ is attached to the level bar and is adjustable at one end and in a vertical direction only, while the other end is fastened permanently by a hinge; a lateral adjustment of the level tube is unnecessary. The telescope itself and the other parts of the instrument are the

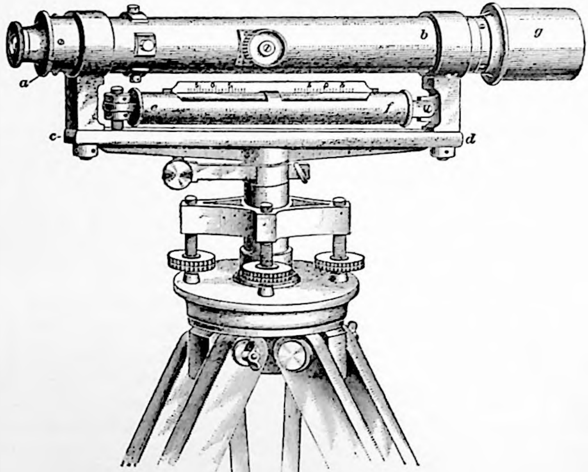


Fig. 9

same as in a wye level and require no special description. The sunshade $g$ is shown in position on the telescope.

**29. Adjustments.**—There are two adjustments of the dumpy level:

1. To make the axis of the level tube perpendicular to the vertical axis of rotation, so that, when the instrument is leveled up, the bubble will remain in the center of the level tube as the telescope is revolved.

2. To make the line of sight parallel to the axis of the level tube, so that the line of sight will be horizontal when the bubble stands in the center of the tube.

There is also the preliminary adjustment to make the cross-wires horizontal and vertical.

**30.** The preliminary adjustment is exactly the same as described for the wye level in Art. **22**. The first adjustment is performed in the same manner as the third adjustment of the wye level, explained in Art. **25**. However, in this case, the capstan screws, shown in Fig. 9 at the left-hand end of the level tube, are used instead of those at the ends of the level bar in Fig. 2.

For the second adjustment, the peg method is employed. The level is set up at $C$, Fig. 8, and rod readings $r_1$ and $r_2$ are taken on the pegs at $A$ and $B$, as described in Art. **27**. Then the level is moved to $D$ and rod readings $r_3$ and $r_4$ are taken on the pegs. If the horizontal cross-wire is properly centered, the values of $r_2-r_1$, or $d_1$, and $r_4-r_3$, or $d_2$, will be equal.

If $d_1$ and $d_2$ are not equal, the cross-wire is adjusted by the following method. Let $m\,p$ represent a horizontal line through $m$. Then the right triangles $n\,t\,k$ and $n\,m\,p$ are similar, since the acute angles at $n$ are equal. Hence, the corresponding sides are proportional and $\frac{n\,k}{n\,p}=\frac{k\,t}{pm}$. From the figure, $n\,k=e$, $n\,p=r_4-r_3-d_1=d_2-d_1$, $k\,t=B\,D=A\,B+A\,D$, and $p\,m=A\,B$.

Therefore,
$$\frac{e}{d_2-d_1}=\frac{AB+AD}{AB}$$

or
$$e=\frac{AB+AD}{AB}(d_2-d_1)$$

In case $d_2$ is less than $d_1$, the difference $d_1-d_2$ is taken instead of $d_2-d_1$.

To make the adjustment, keep the level at $D$ with the bubble centered in the tube. Then, move the horizontal wire by means of the capstan-headed screws until the reading on the rod at $B$ is either $r_4+e$ or $r_4-e$, according to the following rules:

tion, revolve the telescope through a half revolution on the vertical axis of the instrument so that it is over the same pair of screws but is reversed end for end. If the bubble stays in the center of the tube, the wyes do not need adjustment. But if the bubble leaves the center of the tube, bring it half way back by means of the capstan-pattern nuts at the ends of the level bar. Then center the bubble accurately by means of the leveling screws and test the adjustment by again turning the telescope through a half revolution. Repeat the operations until the bubble remains in the center for both positions of the telescope. As a check, try the reversal over the other pair of leveling screws. If the bubble is in adjustment over one pair of screws, it should be correct over the other pair also; any difference is the fault of the instrument and cannot be adjusted.

**26. Tests for Adjustment of the Wye Level.**—The adjustments of the wye level can be checked by the following two tests.

Set up the instrument at any convenient point and level the telescope over both pairs of opposite leveling screws.

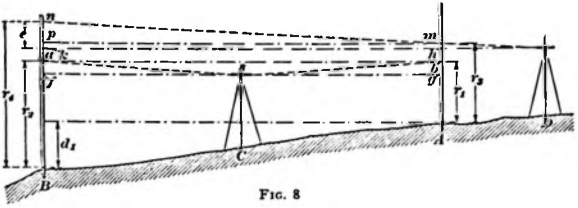


Fig. 8

Set a point at either end of the horizontal cross-wire and then slowly revolve the telescope on the vertical axis so that the point appears to move along the wire. If the point stays on the wire, the wire is horizontal. This test checks the preliminary adjustment described in Art. 22.

**27.** The following test is known as the *peg method* and indicates whether the line of sight is parallel to the axis of the level tube. There are several variations of the method, but only the one most commonly used is given. Select a

stretch of nearly level ground and drive two pegs at $A$ and $B$, Fig. 8, about 400 feet apart. Set up the instrument at $C$, midway between them and as nearly as possible on line with them; but great accuracy in line or distance is not required. With the bubble centered accurately, take readings on leveling rods held on the pegs at $A$ and $B$. Leveling rods and the methods of taking these readings will be described later. If these readings are denoted by $r_1$ and $r_2$, the exact difference in elevation between the pegs is $r_2-r_1$, or $d_1$, no matter how much out of adjustment the instrument may be. If the level is in adjustment, the lines of sight will be horizontal along $s\,f$ and $s\,g$. If the instrument is not in adjustment, the lines of sight will be, say, along $s\,a$ and $s\,b$; but the angles $a\,s\,f$ and $b\,s\,g$ are equal. Since the distances $AC$ and $BC$ are very nearly equal, the differences on the rod, $a\,f$ and $b\,g$, can be considered equal; hence, $a\,b$ is parallel to $f\,g$ and is also horizontal. Therefore, the same difference in elevation between the pegs at $A$ and $B$ is obtained whether the lines of sight are along $s\,f$ and $s\,g$, or along $s\,a$ and $s\,b$.

Next, move the instrument to $D$, about 10 feet beyond $A$ on line with $A$ and $B$ (a point beyond $B$ would serve just as well), and again take rod readings on the pegs at $A$ and $B$. If the readings are denoted by $r_3$ and $r_4$, the difference in elevation between the pegs, as determined by these readings, is $r_4-r_3$, or $d_2$. If the instrument is in adjustment, the values of $d_1$ and $d_2$ will be equal; that is, the line of sight is along the horizontal line $h\,k$. If the instrument is not in adjustment, the line of sight will be represented by some line, such as $m\,n$, which is not horizontal. Then $d_1$ and $d_2$ will not be equal, and the difference may be due to inaccuracy in the first adjustment, in the second adjustment, or perhaps in both. Therefore those adjustments should be repeated

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### THE DUMPY LEVEL

**28. Description.**—A dumpy level is shown in Fig. 9. In its general construction it is similar to the wye level. However, there are two important differences: First, in the

dumpy level, the telescope $a\ b$ is attached rigidly to the level bar $c\ d$. Second, the level tube $c\ f$ is attached to the level bar and is adjustable at one end and in a vertical direction only, while the other end is fastened permanently by a hinge; a lateral adjustment of the level tube is unnecessary. The telescope itself and the other parts of the instrument are the

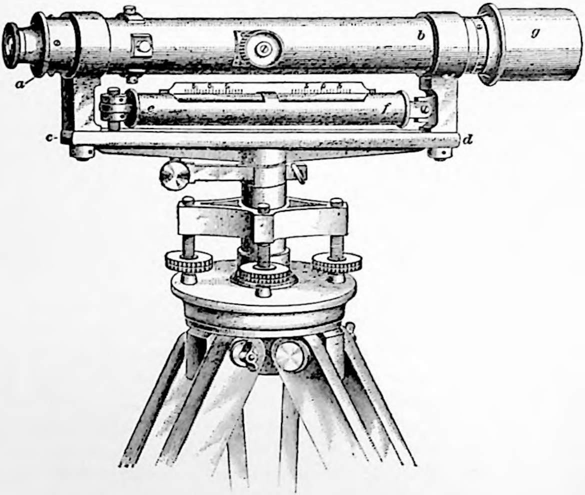


FIG. 9

same as in a wye level and require no special description. The sunshade $g$ is shown in position on the telescope.

**29. Adjustments.**—There are two adjustments of the dumpy level:

1. To make the axis of the level tube perpendicular to the vertical axis of rotation, so that, when the instrument is leveled up, the bubble will remain in the center of the level tube as the telescope is revolved.

2. To make the line of sight parallel to the axis of the level tube, so that the line of sight will be horizontal when the bubble stands in the center of the tube.

There is also the preliminary adjustment to make the cross-wires horizontal and vertical.

**30.** The preliminary adjustment is exactly the same as described for the wye level in Art. **22.** The first adjustment is performed in the same manner as the third adjustment of the wye level, explained in Art. **25.** However, in this case, the capstan screws, shown in Fig. 9 at the left-hand end of the level tube, are used instead of those at the ends of the level bar in Fig. 2.

For the second adjustment, the peg method is employed. The level is set up at $C$, Fig. 8, and rod readings $r_1$ and $r_2$ are taken on the pegs at $A$ and $B$, as described in Art. **27.** Then the level is moved to $D$ and rod readings $r_3$ and $r_4$ are taken on the pegs. If the horizontal cross-wire is properly centered, the values of $r_2-r_1$, or $d_1$, and $r_4-r_3$, or $d_2$, will be equal.

If $d_1$ and $d_2$ are not equal, the cross-wire is adjusted by the following method. Let $m\,p$ represent a horizontal line through $m$. Then the right triangles $n\,t\,k$ and $n\,m\,p$ are similar, since the acute angles at $n$ are equal. Hence, the corresponding sides are proportional and $\frac{n\,k}{n\,p}=\frac{k\,t}{p\,m}$. From the figure, $n\,k=e$, $n\,p=r_4-r_3-d_1=d_2-d_1$, $k\,t=B\,D=A\,B+A\,D$, and $p\,m=A\,B$.

Therefore,
$$\frac{e}{d_2-d_1}=\frac{A\,B+A\,D}{A\,B}$$

or
$$e=\frac{A\,B+A\,D}{A\,B}(d_2-d_1)$$

In case $d_2$ is less than $d_1$, the difference $d_1-d_2$ is taken instead of $d_2-d_1$.

To make the adjustment, keep the level at $D$ with the bubble centered in the tube. Then, move the horizontal wire by means of the capstan-headed screws until the reading on the rod at $B$ is either $r_4+e$ or $r_4-e$, according to the following rules:

**Rule I.**—*If $d_1$ is less than $d_2$ and $r_3$ is greater than $r_4$, or if $d_1$ is greater than $d_2$ and $r_3$ is less than $r_4$, add $e$ to $r_4$.*

**Rule II.**—*If $d_1$ is greater than $d_2$ and $r_3$ is greater than $r_4$, or if $d_1$ is less than $d_2$ and $r_3$ is less than $r_4$, subtract $e$ from $r_4$.*

It is to be remembered that $r_3$ is always the reading on the peg nearer the second set-up and $r_4$ is the reading on the peg farther from the set-up. Also, the correction $e$ is always applied to $r_4$. If $e$ is added to $r_4$, the upper capstan screw is loosened and the lower tightened for either an erecting or an inverting telescope. If $e$ is subtracted from $r_4$, the lower screw is loosened and the upper screw tightened. The adjustment should be tested by taking new observations for $r_3$ and $r_4$; if $r_4 - r_3$ is not equal to $d_1$, the wire must be readjusted.

When the difference between $d_1$ and $d_2$ is very small, say less than .02 foot, the following method of determining the reading at which the wire is to be set may be used. Make the rod reading at $B$ equal to $r_3 + d_1$ or $r_3 - d_1$ according to whether $r_4$ is greater or less than $r_3$.

Example 1.—With the instrument at $C$, Fig. 8, rod readings on pegs at $A$ and $B$, 450 feet apart, are, respectively, $r_1 = 3.718$ feet and $r_2 = 5.142$ feet. When the instrument is moved to $D$, 9 feet behind $A$, the rod readings on the pegs are $r_3 = 6.005$ feet at $A$, and $r_4 = 7.470$ feet at $B$. (*a*) Is the instrument in adjustment? (*b*) At what reading should the cross-wire be set to make the adjustment?

Solution.—(*a*) The true difference in elevation between the pegs is $d_1 = r_2 - r_1 = 5.142 - 3.718 = 1.424$ ft. The difference in elevation given by the readings from $D$ is $d_2 = r_4 - r_3 = 7.470 - 6.005 = 1.465$ ft. Since $d_1$ and $d_2$ are not equal, the instrument is not in adjustment. Ans.

(*b*) To determine the correction $e$, substitute in the formula the values $A\ B = 450$, $A\ D = 9$, $d_1 = 1.424$, and $d_2 = 1.465$; then,

$$e = \frac{450+9}{450}(1.465 - 1.424) = .042 \text{ ft.}$$

Since $d_1$ is less than $d_2$ and $r_3$ is less than $r_4$, rule II applies. Hence, $e$ is subtracted from $r_4$ and the required rod reading is $7.470 - .042 = 7.428$ ft. Ans.

Example 2.—Suppose that, for the pegs in example 1, the level is set up 9 feet behind $B$ and the rod readings are 4.836 feet at $B$ and 3.422 feet at $A$. At what reading should the cross-wire be set to adjust the instrument?

SOLUTION.—As in example 1, $d_1 = 1.424$ ft. In this case, however, the peg at $B$ is nearer the second set-up, and the peg at $A$ is farther from the set-up; hence, $r_3$, which is the rod reading at $B$, is 4.836 ft. and $r_4$ at $A$ is 3.422 ft. The difference $d_2$ is $4.836 - 3.422 = 1.414$ ft. and $d_1 - d_2 = 1.424 - 1.414 = .010$ ft., which is less than .02 ft.

Since $r_4$ is less than $r_3$, $d_1$ is subtracted from $r_3$ and the required rod reading is $r_3 - d_1 = 4.836 - 1.424 = 3.412$ ft. Ans.

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**EXAMPLES FOR PRACTICE**

1. In adjusting a dumpy level, the instrument is set up midway between two pegs $A$ and $B$, 400 feet apart, and the rod readings on the pegs are 4.172 feet at $A$ and 2.065 feet at $B$. Then the level is set up 10 feet behind $B$, and the readings are 6.103 feet at $B$ and 8.155 feet at $A$. What should be the reading at $A$ for adjusting the cross-wire? Ans. 8.211 ft.

2. For the same two pegs as in example 1, the level is set up 8 feet behind $A$, and the rod readings are 5.555 feet at $A$ and 3.440 feet at $B$. At what reading should the cross-wire be set? Ans. 3.448 ft.

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## LEVELING RODS

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### DESCRIPTION

**31. Principle of Direct Leveling.**—When an adjusted level is set up and leveled up, its line of sight is horizontal; and, since the line of sight is at right angles to the vertical axis of the instrument, it rotates about this axis in a horizontal plane, called the *plane of the instrument*. The elevation of this plane is the elevation of the instrument, usually called *height of instrument*, and every line lying in the plane is a horizontal line having the same elevation as the instrument. This elevation may be assumed arbitrarily or may be determined from the known elevation of some other point by measuring the vertical distance from the plane of the instrument, as defined by the line of sight, to the point of known elevation. A leveling rod is usually employed for this purpose.

The general method of direct leveling is illustrated in Fig. 10. It is assumed that the elevation of point $A$ is 976 feet and it is desired to determine the elevation of point $B$. The level is

set up at some convenient point so that the plane of the instrument is higher than both $A$ and $B$. Then a leveling rod is held vertically at the point $A$, which may be the top of a stake, or some object, and the line of sight is directed toward the rod. The vertical distance from $A$ to the line of sight can be read on the rod at the point cut by the horizontal cross-wire of the telescope. If the rod reading is 8 feet, the line of sight is 8 feet above the point $A$ and the height of instrument is $976+8=984$ feet. Then the leveling rod is held vertically at $B$, and the line of sight is directed toward the rod by revolving the telescope on its vertical axis. The vertical distance from $B$ to the line of sight is given by the rod reading with which the cross-wire coincides. If the rod reading at $B$ is 3 feet, it means that the point $B$ is 3 feet below the line of sight and, therefore, the elevation of $B$ is $984-3=981$ feet.

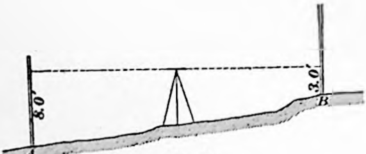


FIG. 10

**32. Kinds of Leveling Rods.**—In general, a leveling rod is a graduated wooden rod. There are several kinds of rods differing in constructive details but not in principle. The two types most commonly used are known as the *Philadelphia rod* and the *New York rod*, the important difference being in the method of marking the graduations.

**33. Philadelphia Rods.**—The Philadelphia rod, Fig. **11** or **12**, is made in two sections, held together with brass sleeves *a* and *b*. The rear section slides with respect to the front section, and it can be held in any desired position by means of the clamp screw *c* on the upper sleeve *b*. In Fig. **13** (*a*) is shown a side view of part of a rod. The upper portion $A$ of the rear section projects so that its face is in line with that of the front section $B$. When the projection rests on the lower section, as shown in Fig. **13** (*b*), the rod is said to be closed; a closed rod is sometimes called *short rod*. When the

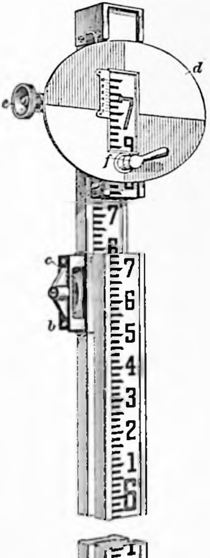


Fig. 11

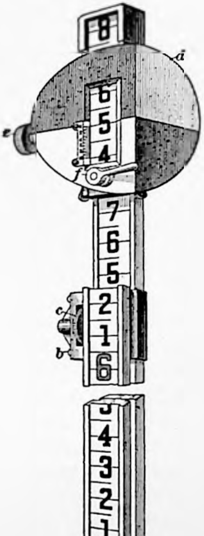
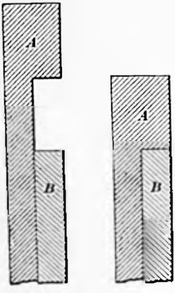


Fig. 12

rod is extended, no matter how much, it is known as *long rod*, or *high rod*.



(a) (b)

Fig. 13

In the type shown in Fig. 11, the divisions are alternate black and white spaces, each .01 foot high, painted on the rod. Each fifth hundredth is indicated by a longer graduation mark, so that an acute angle is formed at one corner of the black space of which the graduation is a part. The tenths are marked by large black figures, half above and half below the graduation mark, and the feet are shown in a similar manner by red figures (shaded in the illustrations). The graduations can be seen distinctly through the telescope of a level at a great distance, and, therefore, Philadelphia rods are sometimes called *self-reading rods* or *speaking rods*.

In the type of a Philadelphia rod shown in Fig. 12, the graduations indicate tenths of a foot and half-tenths. They are marked by single lines, and are numbered as in Fig. 11.

The projection at the top of the sliding portion of the rod is graduated upwards from the greatest value on the lower section so that, when the rod is closed, the graduations are continuous; they run to 7 feet for the type in Fig. 11 and somewhat above 6.5 feet for the rod in Fig. 12. The front of the sliding portion of the rod, which is hidden when the rod is closed, is also graduated upwards from the greatest value on the lower portion so that, when the rod is fully extended, the graduations are again continuous as shown in Fig. 14. The rod in Fig. 11 extends to 13 feet while the highest reading for the rod in Fig. 12 is 12 feet. The back of

Fig. 14

the sliding portion of the rod is also graduated; the purpose and the arrangement of these graduations will be described later.

**34.** In case the graduations cannot be read directly from the telescope, the device shown at *d*, Fig. 11 or 12, called a *target*, is used. The target is a circular or elliptical metal plate divided into quadrants alternating red and white. The target in Fig. 11 is a plane surface but that in Fig. 12 consists of two plane surfaces bent at right angles to each other. When the rod is held vertical, one of the lines dividing the colors is horizontal and the other is vertical. There is an opening in the face of the target in order that the graduations on the face of the rod can be seen through it. One side of this opening is beveled to a thin edge, and a scale is marked along this edge so that it is close to the face of the rod. This scale is used for determining readings between graduation marks. One of its ends is exactly on the horizontal line dividing the colors on the target, and it extends either entirely above this line, as in Fig. 11, or entirely below, as in Fig. 12.

When the clamp screw *e* is loose, the target can be moved over the face of the rod until the line dividing the colors coincides with the horizontal cross-wire of the level. The target can then be fixed in that position by tightening the clamp *e*. The small lever *f* is used to move the target a short distance after the clamp *e* has been tightened. If the target is not needed, it can be removed from the rod.

**35.** When the target is used on a high rod, it is first set exactly at 7 feet on the extension part of the rod as shown in Fig. 11, or at 6.5 feet as in Fig. 12. Then the extension with the target is raised until the line dividing the colors coincides with the horizontal wire of the telescope, and the rod is held in that position by tightening the clamp *c*. In order to indicate the reading for a high rod, the graduations on the back of the sliding part and a scale on the back of the upper sleeve are used. The back of a rod of the type in Fig. 11 is shown in Fig. 15. The graduations begin at 7 feet at the top and increase


FIG. 15

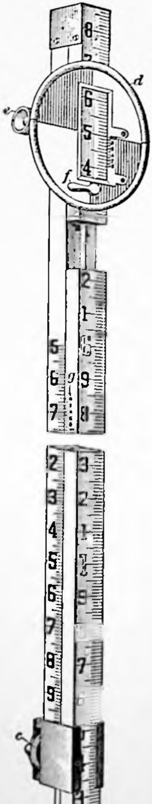



FIG. 16

downwards to 13 feet. The reason for this is given by the following explanation:

The reading of a high rod is the distance from the base of the rod to the target. Thus, for the position shown in Fig. 15, the rod reading represents the distance $h$. When the target is set at 7 feet on the extension part of the rod while the rod is closed, the reading of the rod is 7 feet and the 7-foot graduation on the back of the rod is opposite the zero of the scale on the sleeve. As the rod is raised, the 7-foot mark moves upwards, while the zero mark of the scale remains stationary since it is attached to the lower portion of the rod. The distance between these two marks, $d$ in Fig. 15, therefore, increases as the rod is extended farther. Consequently, the distance $d$ is equal to the amount by which the target is raised above 7 feet, and, for any high-rod setting, the distance $h$ is equal to 7 feet plus the distance $d$. To obviate actual addition, the foot-graduations on the back of the rod are numbered downwards from 7 to 13; thus, when the rod is extended 1 foot, the reading on the back is $7+1=8$ feet, and so on. It is, therefore, seen that the numbers must increase downwards in order that the rod readings may become greater as the target is raised.

On the rod shown in Fig. 12, the graduations on the back of the rod begin at 6.5 feet at the top and increase downwards to 12 feet.

**36. New York Rods.**—The New York rod, Fig. 16, is in two sections that slide on each other by means of a tongue and groove; the upper section can be clamped in any position by the clamp screw $c$. The New York rod differs from the Philadelphia rod, first, in the character of the graduation marks, which are single lines stamped on the rod and blackened, much the same as on an ordinary rule. All New York rods are graduated to hundredths of a foot. Usually, the rod is graduated to 6.5 feet on the lower portion and extends to 12 feet. Another difference from the Philadelphia rod is that the inner face of the extending portion of a New York rod is not graduated. Therefore, the readings of an extended rod cannot be taken directly through the telescope. For determin-

ing the reading when the rod is extended, the rear section is graduated along one edge from 6.5 to 12 feet, increasing downwards, and an auxiliary scale $g$ is marked on the edge of the lower portion. Since the graduations are not easily seen, a target is almost always necessary in using a New York rod.

**37. Other Rods.**—Sliding rods are also made in three sections, having a length of about 4.5 feet when closed and extending to 12 feet, or having a length of 5.5 feet when closed and extending to 15 feet. Sliding rods are objectionable because they sometimes stick or slip when extended. Therefore, some rods are made in one piece 10 or 12 feet long, and others are in two parts that are connected by a hinged joint and that fold together. All these rods are self-reading, and a target cannot be used on them. They are graduated only on the front face of each section, and when the rod is fully extended, the graduations are continuous.

For work in mines and tunnels and for other purposes, it is sometimes convenient to use a very short rod. Therefore, two-section rods are made which are 3 feet long when closed and extend to 5 feet.

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### READING THE RODS

**38. Rodman.**—The man who carries the rod and holds it on the points whose elevations are to be taken is called a rodman. A good rodman is essential to accurate and rapid leveling. A man who is slow and inattentive to the work is not suitable for a rodman. In most localities, a line of levels of any considerable length will have enough rough places in it—that is to say, places where abrupt and considerable changes in elevation occur—to retard progress, however diligent the level party may be. The laziness or carelessness of an individual should never be allowed to delay the progress of the party.

**39. Using the Rod.**—A rod can be held on a point and carried more easily when it is closed; therefore, it is usually extended only when the reading exceeds the highest graduation on the lower section. Thus, with a rod like that shown in Fig.

11, short-rod readings may be taken up to 7 feet, and with the styles shown in Figs. 12 and 16, the highest reading on the short rod is 6.5 feet. The readings may be made either with or without the aid of the target *d*. In all cases, the rod is held with the front toward the level. When the target is not used, the levelman reads the position of the horizontal cross-wire directly from the telescope. If the target is used on a short rod, the rodman moves it up or down as indicated by word or signal from the levelman until the line separating the colors nearly coincides with the horizontal wire. The target is then clamped by tightening the screw *e*, and its dividing line is set exactly on the wire by means of the lever *f*. The reading of the rod is determined by the position of the end of the target scale which is on the line between the colors.

To make a high-rod reading without the target, the rod shown in Fig. 11 or 12 is extended to its full length. Then the graduations on the front appear continuous and the reading of the horizontal wire can be taken from the telescope. If the target is used on a high rod, it is first set exactly at 7 feet on the extension part of the rod shown in Fig. 11, or at 6.5 feet for the rods shown in Figs. 12 and 16. Then the rod is extended until the line dividing the colors on the target coincides with the horizontal wire, and is clamped in that position by tightening the screw *c*. The reading of the rod is indicated by the zero of the scale on the upper sleeve of the rod shown in Fig. 11 or 12, or on the side of the rod shown in Fig. 16.

**40. Reading the Rod Directly.**—If a target is not used, the reading of a rod is made directly from the telescope in the following manner. The number of feet is given by the shaded figure (red on the actual rod) below the horizontal cross-wire. The number of tenths is shown by the black figure directly below the wire. If the reading is required to the nearest hundredth on the rods shown in Figs. 11 and 16, the number of hundredths is found by counting the divisions between the last tenth and the graduation mark nearest to the wire. If thousandths of a foot are required, the number of hundredths

is equal to the number of divisions between the last tenth and the graduation mark below the wire, and the number of thousandths is estimated by judgment. For example, the readings on the Philadelphia rod for the positions $x$, $y$, and $z$ in Fig. 17 ($a$) are determined as follows: For $x$, the number of feet below is 4 and the number of tenths below is 1; the wire coincides with the first graduation above the tenth-mark and, consequently, the reading is 4.11 feet to the nearest hundredth, or 4.110 feet to the nearest thousandth. For $y$, the feet and tenths are again 4 and 1, respectively. The wire is just midway between the graduations indicating 4 and 5 hundredths, and, therefore, the reading to the nearest hundredth may be taken as either 4.14 or 4.15 feet; in determining the hundredths, it is convenient to observe that the wire is just below the acute-angle graduation denoting the fifth hundredth, and it is, therefore, unnecessary to count up from the tenth-graduation. If thousandths are required, the following method is used for finding the hundredths and thousandths: There are 4 divisions between the tenth-mark and the graduation below the wire; hence, the number of hundredths is 4. Since the wire is midway between the two graduation marks on the rod, and since the distance between graduations is 1 hundredth or 10 thousandths of a foot, the number of thousandths in the required reading is $\frac{1}{2} \times 10$, or 5; hence, the reading to the nearest thousandth is 4.145 feet. For $z$, the foot just above is 4 and, therefore, the foot below must be 3 (not shown in the figure); the number of tenths is evidently 9 and the number of hundredths 6; the distance to the wire from the hundredth mark below is about one-third of a graduation, as nearly as can be estimated, and, consequently, the number of thousandths is

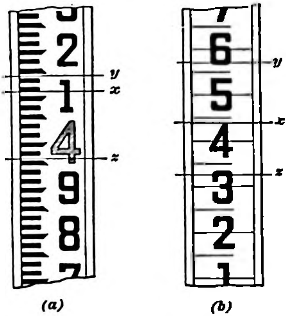



Fig. 17

$\frac{1}{3}\times 10$, or 3; hence, the reading to the nearest hundredth is 3.96 feet and, to the nearest thousandth, 3.963 feet. A New York rod may be read in the same way.

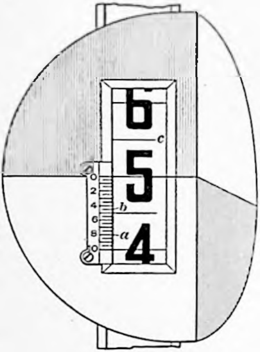
**41.** For the Philadelphia rod shown in Fig. 12, the feet and tenths are obtained as just explained for the other rod, and the hundredths and half-hundredths may be estimated with the aid of the half-tenth graduations. Thus, the readings of the cross-wire for the positions *x*, *y*, and *z* in Fig. 17 (*b*) are found as follows: Assume that the foot mark below the part of the rod shown is 5. For *x*, the number of tenths is 4, and the number of hundredths is less than 5 since the wire is below the half-tenth (.05) graduation; the distance above the tenth-mark is estimated to be about $\frac{4}{5}$ of the distance between graduations and the number of hundredths is, therefore, $\frac{4}{5}\times 5=4$; hence, the reading is 5.44 feet. For *y*, the number of tenths is 5 and the number of hundredths is more than 5; the distance above the half-tenth graduation is estimated to be $\frac{2}{5}$ of the distance to the tenth-mark and, consequently, the number of hundredths is $5+\frac{2}{5}\times 5=7$; the reading is, therefore, 5.57 feet. For *z*, the wire is midway between the tenth and half-tenth graduations and its distance above the mark that indicates 3 tenths is $\frac{1}{2}\times .05=.025$; hence, the rod reading is 5.325 feet.

Direct readings on a long rod are made in the same way as on a short rod because the rod is fully extended and the graduations appear continuous.

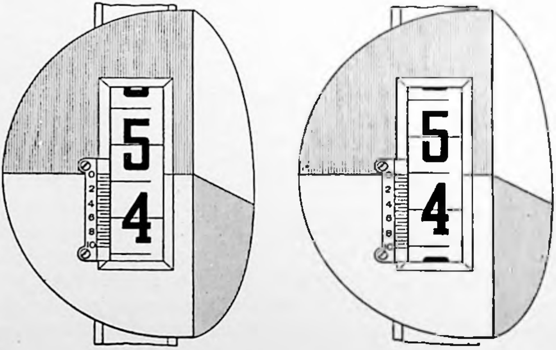
**42. Scales.**—In order to aid in reading a rod, a target is sometimes used. When the rod is graduated only to tenths or half-tenths of a foot, the scales on the target and on the upper sleeve are made exactly .1 foot long and are divided into 10 or 20 parts—that is, to hundredths or half-hundredths of a foot—so that hundredths or half-hundredths can be read directly and thousandths can be estimated with a fair degree of accuracy.

In Fig. 18 are shown settings of the target on the rod illustrated in Fig. 12; the zero point of the scale coincides with the line dividing the colors on the target and indicates the position where the cross-wire cuts the rod. The length of the scale is

.1 foot and, since it is divided into 20 parts, each small division is $\frac{1}{20}\times .1 = .005$ foot. The numbers on the scale indicate hundredths of a foot, the distance from 0 to 2 being .02 foot, from 0 to 4, .04 foot, etc. Thus, in (*a*), the distance of the point *a* below the zero mark is .08 foot and the distance of the point *b* below zero is .045 foot. It will be observed that the numbers on the scale increase downwards. Since the zero of the scale coincides with the cross-wire of the telescope, the reading of the rod consists of two parts: The feet and tenths are given on the rod by the




(*a*)




(*b*) (*c*)

Fig. 18

nearest numbers below the zero of the scale; the remainder of the reading is the distance measured on the scale from the zero

of the scale to the nearest lower tenth-graduation on the rod. It should be noticed that, when the target is used, the half-tenth graduations on the rod, as at *c*, are disregarded.

For the setting in (*b*), the zero of the scale is above the graduation 4 on the rod; if the next lower foot on the rod is assumed to be 5, the feet and tenths for the reading are 5 and 4, respectively. The distance from the graduation 4 on the rod to the zero of the scale is shown to be .06 foot, since graduation number 6 on the scale is opposite that graduation on the rod. Hence, the reading is $5.4+.06=5.46$ feet.

In case no graduation of the scale coincides exactly with a graduation on the rod, the thousandths are estimated by judgment. Thus, in Fig. 18 (*c*), the graduation 4 on the rod is slightly below a point midway between the marks on the scale indicating .045 and .05, and the distance from the graduation 4 on the rod to the zero point on the scale is estimated to be .048. Hence, the reading of the target is taken as 5.448 feet.

**43.** When a target is used on a high rod, the reading is made by means of the scale on the back of the upper sleeve or on the side of the rod according to the type of the rod. In Fig. 19, *a b* represents part of the back of the rod shown in Fig. 12, and *c d* is the scale on the upper sleeve. When the rod is closed, the zero of the scale coincides with the graduation on the rod indicating 6.5 feet, as shown in (*a*), because the target is set at 6.5 feet when the rod is extended. As the upper section of the rod is raised, the scale remains stationary, since it is attached to the lower section, but the graduations on the back of the rod move. The numbers of these graduations increase downwards in order that the reading should become greater as the target is raised. The numbers on the scale, therefore, increase upwards.

The method of reading a long rod is similar to that for a short rod, the difference being that the feet and tenths are given by the numbers above the zero of the scale instead of below it; for instance, in Fig. 19 (*b*) the number of feet is 10 and the number of tenths is 1. The hundredths and thou-

sandths are found from the position of the scale with respect to the tenth-graduation on the rod just as for a short rod; since the mark indicating 7 hundredths coincides with the graduation 1 on the rod, the reading is 10.17 feet.

In (*c*), the feet and tenths are again 10 and 1, respectively, but no graduation on the scale coincides with the graduation on the rod. The graduation 1 is between the graduations on the scale indicating .045 and .05 and the distance from 1 on the rod to zero on the scale is estimated to be .047 foot; hence, the reading is taken as 10.147 feet.

**44. Principle of the Vernier.**—When the rods in Figs. 11 and 16 are read directly from the telescope, it is necessary to

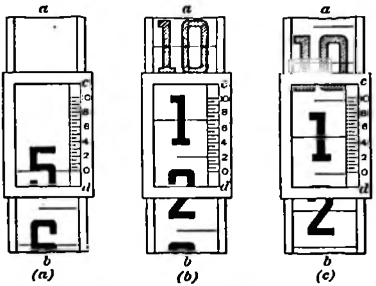


FIG. 19

estimate thousandths of a foot. When the target is used, thousandths may be read accurately by means of the scale on the target, which is called a vernier. A vernier is a movable auxiliary scale used for the purpose of measuring accurately, on another scale, fractional parts of the smallest subdivisions of the main scale. Thus, if the main scale is divided into feet and tenths of a foot, the vernier may be used to measure on the main scale hundredths of a foot; or if the main scale is divided into tenths and hundredths of a foot, the vernier may be used to measure thousandths of a foot. As verniers are used extensively on surveying instruments, the principles

on which their construction is based should be thoroughly understood.

To illustrate the fundamental principle of a vernier, suppose that the bar $a\,b$, Fig. 20, is to be measured with the scale

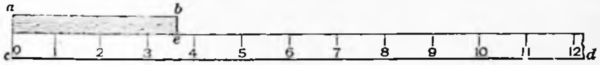



FIG. 20

$c\,d$, which is divided in inches. The scale is placed against the bar so that the zero of the scale coincides with one end of the bar, and the position of the other end of the bar is observed. In this case, the bar measures 3 inches plus the distance between the graduation mark 3 and the point $e$ which is opposite the end of the bar. Suppose now that it is desired to measure the distance 3-$e$ in eighths of an inch. For this purpose a vernier

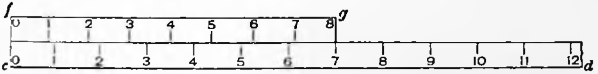



FIG. 21

$f\,g$, Fig. 21, is prepared, which has a total length of 7 inches and is divided into eight equal parts. Each division is thus equal to $\frac{7}{8}$ inch and the difference between one division of the main scale and one division of the vernier is $1-\frac{7}{8}=\frac{1}{8}$ inch. Although distances in surveying are generally measured in feet and decimals of a foot rather than in feet, inches, and fractions of an inch, inches and eighths are used here to demonstrate the general principle of a vernier.

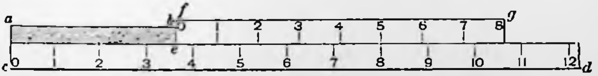



FIG. 22

To measure the distance 3-$e$ with the vernier, place the vernier against the main scale, as shown in Fig. 22, so that the zero of the vernier coincides with the point $e$. To find the number of eighths in the distance 3-$e$, look for a graduation

mark on the vernier that coincides with some graduation on the main scale. In this case it will be noticed that graduation number 5 of the vernier coincides with a mark on the main scale; thus, the number of eighths in 3-*e* is 5, and the distance 3-*e* is, therefore, $\frac{5}{8}$ inch. The explanation of this is as follows.

Imagine that the vernier is placed against the main scale so that the zero mark of the vernier coincides with line 3 of the main scale, and then the vernier is slid along until its zero reaches the point *e*. The distance covered by the zero mark of the vernier during this motion will measure the distance 3-*e* on the bar. When, in this motion, the vernier is in the

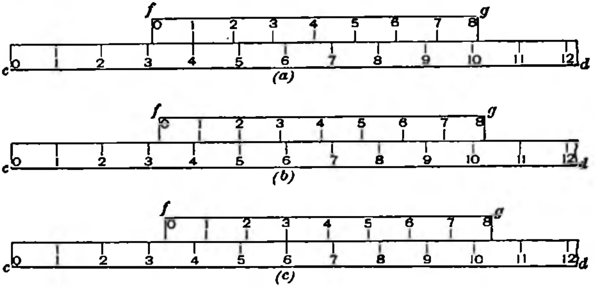



FIG. 23

position shown in Fig. 23 (*a*), the graduation mark 1 coincides with a graduation on the main scale; and the zero mark of the vernier has covered a distance equal to the difference between one division of the main scale and one division of the vernier, or $\frac{1}{8}$ inch. When the vernier is in the position shown in (*b*), point 2 coincides with a point on the main scale; the zero of the vernier has, therefore, moved a distance equal to the difference between two divisions of the main scale and two divisions of the vernier, which is $\frac{2}{8}=\frac{1}{4}$ inch. When in the position shown in (*c*), the zero of the vernier has moved $\frac{3}{8}$ inch; and, finally, when the zero point of the vernier has moved to the point *e*, Fig. 22, it has covered the distance equal to the difference between 5

divisions of the main scale and 5 divisions of the vernier, or $\frac{5}{8}$ inch.

The same conclusion can be drawn directly by analyzing the position shown in Fig. 22. In this position the distance between point 4 on the vernier and point 7 on the main scale equals the difference between one division of the main scale and one division on the vernier; the distance from point 3 on the vernier to point 6 on the main scale equals the difference between two divisions on the main scale and two divisions on the vernier, and so on; finally, the distance between point 3 on the main scale and the zero mark of the vernier, which is opposite point *e*, is equal to the difference between five divisions of the main scale and five divisions on the vernier, or $\frac{5}{8}$ inch.

**45.** In general, the length of a vernier is made equal to a certain number of the smallest divisions of the main scale. In the example just considered the smallest division of the main scale is 1 inch and the number of divisions covered by the vernier is 7; the total length of the vernier is, therefore, 7 inches. This length is divided into a number of parts which is one more than the selected number of main-scale divisions. In the preceding example the vernier is, therefore, divided into $7+1=8$ equal parts, each being $\frac{7}{8}$ inch long.

The difference between one division of the main scale and one division of the vernier is called the *least reading of the vernier*. It is equal to the length of one smallest division of the main scale divided by the number of parts into which the vernier is divided. In the example just considered, the smallest division of the main scale is 1 inch and the vernier is divided into 8 parts; therefore, the least reading of the vernier is in this case $\frac{1}{8}$ inch.

**46.** To measure the length of a line with a scale and a vernier, the zero mark of the scale is placed at one end of the line and the vernier is slid along the scale until its zero mark is at the other end of the line. Then the length consists of two parts: first, the distance from the zero mark of the main scale to the graduation of the main scale preceding the zero mark of the vernier, which is indicated by the number

of that graduation; second, the distance from the zero mark of the vernier to the preceding main-scale graduation, which is determined by multiplying the least reading of the vernier by the number of the vernier graduation that coincides with a scale graduation. The first distance is sometimes called the *reading of the scale* and the second the *reading of the vernier;* but generally the term *reading of the vernier* is understood to mean the entire distance from the zero mark of the scale to the zero mark of the vernier. For instance, in the case shown in Fig. 22, the distance on the main scale from the zero mark to the graduation preceding the zero mark of the vernier is 3 inches, as indicated by the number 3 at that graduation; the fractional part of a division is $\frac{5}{8}$ inch, which is the product of the least reading of the vernier, or $\frac{1}{8}$ inch, and the number of the vernier graduation that coincides with a scale graduation, or 5; hence, the total length of the bar is $3+\frac{5}{8}=3\frac{5}{8}$ inches.

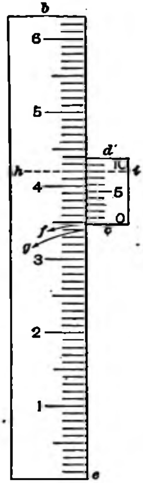


FIG. 24

It should be noticed particularly that in reading the vernier the number of the main-scale graduation, with which the vernier graduation coincides, is of no importance and is not observed. It should also be noticed that the numbers on the vernier increase in the same direction as do those on the main scale.

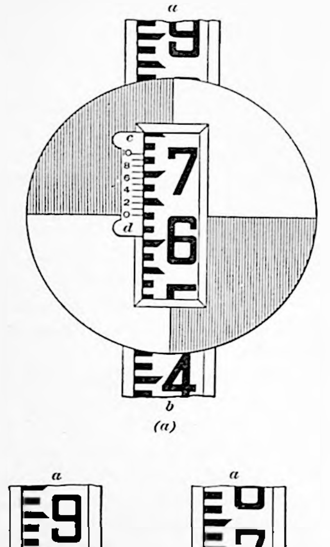
**47. Vernier for Level Rod.**—In Fig. 24 is shown part of a level rod *a b* with a vernier *c d*. The divisions marked 1, 2, etc., on the rod are tenths of a foot, and each of these is divided into ten equal parts, or hundredths of a foot. The vernier covers nine of the smallest subdivisions of the scale and is, therefore, .09 foot long; it is divided into $9+1=10$ parts, each space being one-tenth of .09, or .009 foot. The difference between one division on the rod and one on the vernier is $.01-.009=.001$ foot, which is the least reading of the vernier.

The least reading can also be obtained by dividing the value of a rod division by the number of parts in the vernier; thus, $\frac{.01}{10} = .001$ foot.

The reading of a leveling rod is the distance from the base of the rod to the line dividing the colors on the target, or to the zero of the vernier. This distance consists of two parts, one from the base of the rod to the graduation preceding the zero of the vernier, and the other from that rod graduation to the zero of the vernier. · The first distance is found by the same method as for a direct reading, except that the zero of the vernier takes the place of the cross-wire. The reading of the vernier, which is added, is found by multiplying the least reading of the vernier by the number of the vernier graduation that coincides with a rod graduation.

In Fig. 24, it is assumed that the base of the rod is at $e$ and the reading of the rod is the distance $e\,f$, which is made up of the two parts $e\,g$ and $g\,f$. The first of these is $.3+.04=.34$ foot. To find the distance $g\,f$, it will be noticed that the eighth graduation mark of the vernier coincides with a graduation mark of the rod (it is not necessary to note which mark). This coincidence is indicated by the dotted line $h\,i$. Since one division of the rod is greater by .001 than one division of the vernier, the seventh mark of the vernier is .001 foot from the rod graduation immediately below it; the sixth mark of the vernier is .002 foot from the mark on the rod immediately below it, etc. In this manner it is found that the zero mark of the vernier is .008 foot from the rod graduation $g$ immediately below it. The distance $g\,f$ is, therefore, .008 foot. According to the principle explained in the preceding article, this result may be obtained by multiplying the least reading of the vernier, .001, by 8; thus, $.001 \times 8 = .008$ foot.

The reading of the rod for the target setting in Fig. 25 ($a$) is determined as follows: The number of feet is given by the red figure on the rod $a\,b$ below the zero of the vernier $c\,d$; no foot graduation being shown in the diagram, it will be assumed as 4. The number of tenths is shown by the black figure below the zero of the vernier; in this case, it is 6. The



(a) (b) (c)

Fig. 25

number of hundredths is found by counting the graduations on the rod between the tenth below and the zero of the vernier; the zero of the vernier is above the graduation of the rod indicating .03 foot. Here, the fifth vernier graduation coincides with a rod graduation, and, consequently, the number of thousandths is 5. The reading of the target is, therefore, 4.635 feet.

**48.** When the rear section of the rod is extended, the reading is determined by means of the vernier on the upper sleeve through which the section slides. In Fig. 25 (*b*), *a b* is a portion of the rear section of the rod and *c d* is the vernier on the sleeve. The method of determining the reading is similar to that for the front of the rod, but since the numbers on the rear of the rod increase downwards, the foot, tenth, and hundredth are above the zero of the vernier; the

numbers on the vernier also increase downwards to correspond to the direction in which the numbers on the rod increase. In this case, the foot above the zero of the vernier is 10, the tenth above it is zero, and the hundredth is 5; since the number of the vernier graduation that coincides with a rod graduation is 3, the number of thousandths is also 3; the rod reading is, therefore, 10.053 feet.

In (*c*), the foot below the zero of the vernier is shown as 9 and the foot above will, therefore, be 8; the tenth above the zero of the vernier is 8; the hundredth above the zero of the vernier is 0; and since the seventh graduation of the vernier coincides with a graduation mark on the rod, the number of thousandths is 7. Hence, the reading is 8.807 feet.

The scale on the target and the scale *g* on the side of the rod shown in Fig. 16 are both verniers; and the method of reading the rod is the same as that explained for the rod shown in Fig. 11.

**49. Formulas Relating to Verniers.**—In the following formulas, let

$s$ = length of one of the smallest subdivisions of the scale;
$n$ = number of equal parts into which the vernier is divided;
$r$ = least reading of the vernier;
$l$ = total length of the vernier;
$m$ = number of the vernier graduation coinciding with a graduation of the scale;
$R$ = reading of the vernier.

Then, from the principles explained in the preceding articles,

$$r = \frac{s}{n} \qquad (1)$$

$$n = \frac{s}{r} \qquad (2)$$

$$l = (n-1)s \qquad (3)$$

$$R = r\,m \qquad (4)$$

For example, in Fig. 22, the value $s$ of a scale division is 1 inch; the number $n$ of vernier divisions is 8; the total length $l$ of the vernier is $(n-1)\,s=(8-1)\,1=7$ inches; the least reading $r$ of the vernier is $\frac{s}{n}=\frac{1}{8}$ inch; $m$ is 5, since the fifth vernier graduation coincides with a graduation on the scale; and the reading $R$ of the vernier is $r\,m=\frac{1}{8}\times 5=\frac{5}{8}$ inch.

EXAMPLE 1.—A scale is divided into inches and half inches; the vernier is divided into eight equal parts, which cover seven of the half-inch divisions of the scale. (*a*) What is the least reading of the vernier? (*b*) What is the reading of the vernier when its third graduation coincides with a graduation of the scale?

SOLUTION.—(*a*) Here $s=\frac{1}{2}$ in., and $n=8$. Therefore, by formula **1**,

$$r=\frac{\frac{1}{2}}{8}=\frac{1}{16}\text{ in. Ans.}$$

(*b*) Here $r=\frac{1}{16}$ in. and $m=3$. Therefore, by formula **4**,

$$R=\frac{1}{16}\times 3=\frac{3}{16}\text{ in. Ans.}$$

EXAMPLE 2.—The scale of a barometer is divided into inches and fiftieths. (*a*) What must be the length of a vernier, and how must the vernier be divided, that its least reading may be .002 inch? (*b*) What is the reading of the vernier when its seventh mark coincides with a graduation on the scale?

SOLUTION.—(*a*) In this case, $s=\frac{1}{50}=.02$ in., and $r=.002$ in. Then, by formula **2**,

$$n=\frac{.02}{.002}=10$$

The vernier must, therefore, be divided into ten equal parts covering nine subdivisions of the scale. Its length will be $\frac{9}{50}$ in. Ans.

(*b*) Formula **4** is applied, in which $r=.002$ in. and $m=7$. Hence, $R=.002\times 7=.014$ in. Ans.

**50.** It will be readily seen that a vernier on a leveling rod is practical only when the value $s$ is relatively small, because then the vernier will be of reasonable length. If it were required to use a vernier on the rod in Fig. 12 with a least reading of the vernier of .005 foot, the number of parts of the vernier would be $\frac{.05}{.005}=10$. Then the length of

the vernier would be $(10-1)$ $.05 = .45$ foot, which is about $5\frac{1}{2}$ inches. Since the target would have to be about 12 inches in diameter, a vernier in this case would be impractical.

## OPERATIONS OF DIRECT LEVELING

### GENERAL METHOD

**51. Running a Line of Levels.**—The fundamental principle of direct leveling was illustrated in Fig. 10, in which case the elevation of point $B$ was determined from that of point $A$ by setting up the level between the two points and taking rod readings at $A$ and $B$. In Fig. 26, rods at $A$ and $K$ cannot be seen from the same position of the level. Therefore, if it is required to find the elevation of point $K$ from that of $A$, it is necessary to set up the level several times and to establish intermediate points such as $C$, $E$, and $G$. These are the conditions commonly met with in practice, and may serve as an illustration of the general methods of direct leveling.

Let the elevation of $A$ be assumed as 20.00 feet. The level is first set up at $B$ so that a rod held at $A$ will be visible through the telescope; and the reading of the rod is found to be 8.42 feet. The height of instrument, abbreviated $H.\ I.$, is found by adding this reading to the elevation of point $A$; thus, $20.00+8.42=28.42$ feet $=H.\ I.$ The rod reading at $A$ is taken by directing the line of sight back toward the start of the line and is, therefore, called a *backsight reading*, or simply a *backsight;* backsight is abbreviated $B.\ S.$ A better definition of a backsight, however, is a rod reading which is taken on a point of known elevation to find the height of instrument. Since a backsight is usually added to the elevation of the point on which the rod is held, a backsight is often called a *plus sight*, written $+S$.

After the $H.\ I.$ has been determined by a backsight on $A$, a point $C$ is selected which is slightly below the line of sight, and the reading is taken on a rod held at $C$. If this reading is 1.20 feet, the point $C$ is 1.20 feet below the line of sight, and

the elevation of $C$ is $28.42-1.20=27.22$ feet. This reading is called a *foresight reading*, or *foresight*, which is abbreviated *F. S.*; it is taken on a point of unknown elevation in order to determine that elevation from the height of instrument. Since the reading for a foresight is usually subtracted from the height of instrument to get the elevation of the point on which the rod is held, a foresight is sometimes called a *minus sight* and is written $-S$.

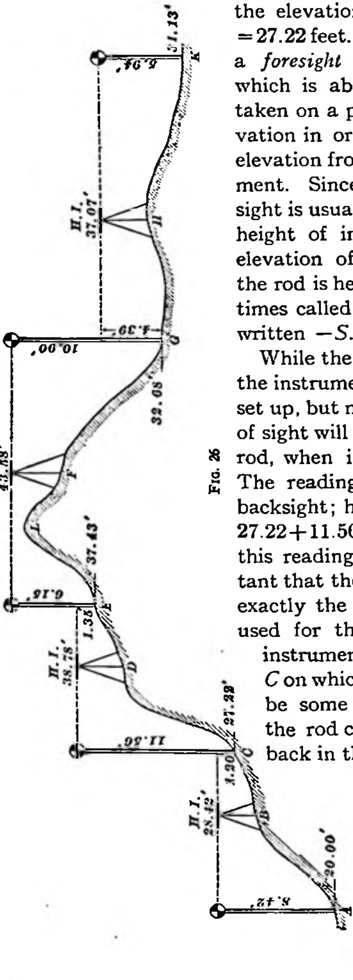


Fig. 26

While the rodman remains at $C$, the instrument is moved to $D$ and set up, but not so high that the line of sight will go over the top of the rod, when it is again held at $C$. The reading 11.56 is taken as a backsight; hence, the *H. I.* at $D$ is $27.22+11.56=38.78$ feet. When this reading is taken, it is important that the rod should be held on exactly the same point that was used for the foresight when the instrument was at $B$. The point $C$ on which the rod is held should be some stable object, so that the rod can be removed and put back in the same place as many times as may be necessary. For this purpose, a sharp-pointed rock, or a well-defined projection on some permanent object, is preferable; but if

nothing better is available, a stake or peg can be driven firmly in the ground and the rod held on top of it. Such a point as $C$, on which both a foresight and a backsight are taken, is called a *turning point*, abbreviated $T.\ P.$

When the $H.\ I.$ at $D$ is known, another turning point $E$ is chosen and a foresight of 1.35 feet is obtained. Then the elevation of $E$ is $38.78-1.35=37.43$ feet. The instrument is set up at $F$ and the backsight on $E$ is 6.15 feet; the new $H.\ I.$ is $37.43+6.15=43.58$ feet. The instrument at $F$ should be high enough to have the line of sight clear the ground at $L$. The rod is held on a turning point at $G$, and since the foresight is 10.90 feet, the elevation of $G$ is $43.58-10.90=32.68$ feet. When the instrument is moved to $H$, the backsight on $G$ is 4.39 feet and the $H.\ I.$ is $32.68+4.39=37.07$ feet. For the rod held at $K$, the foresight is 5.94 feet; the elevation of $K$, therefore, is $37.07-5.94=31.13$ feet. The difference in elevation between $A$ and $K$ is $31.13-20.00=11.13$ feet; that is, point $K$ is 11.13 feet higher than point $A$.

The process of determining the elevations of any series of points is called *running a line of levels.* All the points whose elevations may be determined by a line of levels need not be turning points as is the case in Fig. 26. At each position of the instrument, foresights on any number of points may be taken for the purpose of determining their elevations, either before or after the foresight on the turning point is taken. Points on which only foresights are taken are called *intermediate points.* Thus, the distinction between a turning point and an intermediate point is that both a backsight and a foresight are taken on a turning point, whereas on an intermediate point only a foresight is taken.

EXAMPLE.—The backsight reading on a turning point is 5.28 feet and the foresight reading on the next turning point is 3.25 feet. If the elevation of the first point is 142.00 feet, what are (*a*) the height of instrument and (*b*) the elevation of the second turning point?

SOLUTION.—(*a*) The *H. I.*, which is equal to the elevation of the first point plus the backsight on it, is $142.00+5.28=147.28$ ft. Ans.

(*b*) The elevation of the second turning point is equal to the height of instrument minus the foresight on the turning point; in this case, it is $147.28-3.25=144.03$ ft. Ans.

#### EXAMPLES FOR PRACTICE

1. The backsight reading on a point is 7.36 feet and the foresight reading on a second point is 2.84 feet. If the elevation of the first point is 200.00 feet, what are (*a*) the height of instrument and (*b*) the elevation of the second point?

Ans. { (*a*) 207.36 ft.
(*b*) 204.52 ft.

2. If, when the instrument is set up in a new position, the backsight on the second point mentioned in the preceding example is 11.32 feet, what is the height of instrument? Ans. 215.84 ft.

3. The height of instrument is 125.32 feet and the foresight on a turning point is 4.33 feet. After the instrument has been moved to a new position, the backsight on the turning point is 8.57 feet. What is the elevation of a station, on which the rod reading is 9.20 feet?

Ans. 120.36 ft.

4. The height of instrument is 233.06 feet and the foresight on a turning point is 6.32 feet. After the instrument has been set up in a new position, the backsight on the turning point is 9.58 feet. What are the elevations, to the nearest tenth of a foot, of three stations, on which the rod readings are 5.2 feet, 6.3 feet, and 7.5 feet, respectively?

Ans. { 231.1 ft.
230.0 ft.
228.8 ft.

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#### PRACTICAL CONSIDERATIONS

**52. Plumbing the Rod.**—In order to get the correct vertical distance from the line of sight to the point on which the rod is placed, the rod must be vertical or plumb. When there is not much wind, the rodman can hold the rod practically plumb by balancing it as nearly as possible. Usually, the levelman considers that the rod is plumb in the direction across the line of sight when the edge of the rod is parallel to the vertical cross-wire, and the rodman plumbs the rod in the direction of the line of sight by standing to one side of the rod and judging if it is plumb, or, if possible, comparing it with the vertical edge of a near-by building.

The following method is commonly used for getting an accurate reading especially in a strong wind: The rodman slowly tips the rod backwards and forwards, in the direction of the line of sight and the levelman takes the least reading of the rod as the correct reading. If the target is used, it must

be in such a position that the division line never goes above the horizontal cross-wire but just touches the wire at one position.

The form of a target shown on the rod in Fig. 12 is helpful in plumbing the rod. When the target is properly set and the rod is plumb, the horizontal line dividing the colors on the two faces will appear straight and will coincide with the horizontal cross-wire; while if the rod is not plumb, this line will appear broken. For very accurate work, the rod is plumbed by means of a *rod level,* which carries two spirit levels at right angles to each other, and can be attached to the rod.

**53. Lengths of Sights.**—The most advantageous lengths of sights will depend on the telescope of the instrument, the character of the surface of the country, the sensitiveness of the level bubble, and the degree of accuracy required. In the interests of speed and accuracy, it is always best to have the sights of moderate length; extremely long or short sights should be avoided. For reasonably accurate work, the sights should not usually exceed 400 feet. Where the country is level and time is of great importance, while only an ordinary degree of accuracy is required, quite long sights may be taken. Where the surface rises or falls rapidly, short sights become necessary.

**54.** The most valuable and reliable safeguard against errors due to imperfect adjustment of the instrument is obtained by equal lengths of sights for backsights and foresights on turning points; that is, the distances over which the sights for the backsight and the foresight from the same position of the level are taken should be approximately equal. Should any inequality of length occur in one such pair of sights, it should be balanced in the next pair, or as soon as possible. For example, should the foresight in one pair of sights be taken over a greater distance than the backsight, then in the next pair of sights the distance for the backsight should be made correspondingly longer than that for the foresight. On a hill, the sights on turning points will necessarily be much shorter in one direction than in the other if the level is set

nearly in line with the turning points. In order to make the lengths of sights for the backsight and foresight from a set-up approximately equal, the level may be set to one side of the line. It is not necessary to measure the lengths of the sights accurately; they can be determined closely enough by counting steps in walking.

**55. Bench Marks.**—A permanent point whose elevation is determined in running a line of levels, and which is properly witnessed and recorded for reference, is called a *bench mark*. Any well-defined and easily identified point on a permanent object, such as the door-sill of a building, a stone or concrete monument, the projecting root of a tree, or a point on a large


FIG. 27

rock, will serve for a bench mark. If possible, any line of levels should be started from a bench mark, so that the elevations of the points will be obtained with reference to a definite datum. Bench marks should be established at intervals of from 1,000 to 2,000 feet, depending on the character of the ground and the purpose for which the levels are taken.

To set a bench mark on the root of a tree, a broad notch is cut in the root, as shown at *a* in Fig. 27 (*a*) and (*b*); and a nail is driven almost flush with the surface thus formed. To witness the bench mark, the tree is *blazed* on the side facing the bench mark as shown at *b*. In the blazed space, the letters *B. M.* and the elevation of the bench mark are plainly written with keel, which is a kind of crayon. The rod is held on the nail, which marks a definite point that does not change

in elevation. The elevation of each bench mark is always recorded in the notebook with sufficient description to identify the bench mark unmistakably.

The datum to which the elevation of the bench mark refers may be any imaginary level surface, or it may be sea level, as explained in Art. **6.** Many bench marks whose elevations refer to sea level have been established by the United States Coast and Geodetic Surveys, by the engineers of the various railroad companies, and by other engineers. Whenever practical, a line of levels should be started from such a bench mark in order to refer the elevations of the points to sea level. If the elevation is not written on the bench mark or on its witness, it can usually be obtained from the railroad company or from the engineer who set it. When the elevation of the starting point is assumed, its value should be so much above the datum that in running the line of levels none of the elevations taken will be less than zero. It is customary to make the assumed elevation some multiple of 10 feet, usually 100 feet.

**56. Care in Reading the Rod.**—The rod is always read more closely on bench marks and turning points than on intermediate points or stations. This is because an error in the rod reading on a turning point will affect all subsequent elevations in the line of levels, and an error in the rod reading on a bench mark will affect all later levels that may be referred to the bench mark as a starting point, whereas an error in the rod reading on an intermediate point will affect only the elevation of that point. Consequently, the rod is usually read on an intermediate point only as closely as it is desired to determine the elevation of that point. For grading earth roadways and work of a similar character, the rod is usually read to the nearest tenth of a foot on intermediate points and to the nearest hundredth of a foot on turning points and bench marks: while for work requiring a higher degree of accuracy, such as the surface of a finished pavement, the rod is usually read to the nearest hundredth of a foot on intermediate points and to the nearest thousandth of a foot on turning points and bench marks.

**57. Signals and Calls.**—In running a line of levels it is necessary for the levelman and the rodman to be in almost constant communication with each other. As a means of communication, certain convenient signals and calls are employed. It is important that the levelman and rodman understand these in order to avoid mistakes. When a target reading is taken, the target is set by the rodman in the proper position on the rod, according to signals given by the levelman. An upward movement, or raising, of the hand is the signal for raising the target; a downward movement, or lowering, of the hand is the signal for lowering it; a circle described by the hand is the signal for clamping the target; and a wave with both hands indicates that the target is set properly, or all right. Other signals, such as that for plumbing the rod, are arranged by the members of the party.

The rodman should then read the position of the target on the rod and call out the reading to the levelman, who records it; he should call first the number of feet, or, if the reading is less than 1 foot, he should call naught (not ought); then, after pausing a moment, he should give the decimal part of the reading. Thus, if the rod is being read to hundredths of a foot only, the number 8.40 is called *eight, four, naught;* a reading of 8.04 is called *eight, naught, four.* If the rod is being read to thousandths, the number 8.401 is called *eight, four, naught, one;* 8.410 is called *eight, four, one, naught.* The levelman should always repeat the reading to the rodman in order to insure clear understanding before he records it in the notebook.

When a self-reading rod is used, the rodman keeps the rod extended to full length, unless the country is very level, and the levelman reads the rod on all intermediate points. Then his signal for all right is usually an outward wave of the hand. Sights on turning points or bench marks are often read more closely by use of the target. In many cases, however, the levelman reads the rod on turning points as well as on intermediate points.

In accurate work it is the common practice for the levelman to read the rod without using the target, and to call the reading to the rodman who then sets the target at that value

as a check. In this way, mistakes in feet, which are liable to occur, are avoided, and much time is saved by making it unnecessary to move the target up and down several times before the correct position is found.

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#### FORMS FOR LEVEL NOTES

**58.** Forms of keeping level notes differ somewhat in detail, according to the individual preferences of the engineer, but all are based on the height of instrument. As previously explained, the height of instrument is found by adding a backsight reading to the known elevation of a bench mark or turning point, and the elevation of any point is determined by subtracting a foresight reading from the height of instrument.

The notebook for field notes is ruled with six columns on each page. Three forms of notes in common use will be illustrated and explained.

**59.** In Fig. 28 is shown the method of keeping level notes which is most used. The title or purpose of the survey, the location, the date, and the names and positions of the members of the party should be given. The pages should be numbered. The notes may read downwards from the top of the page, or upwards from the bottom of the page. In the first column of each page are recorded the points, or stations, at which the rod is held; in the second column, the backsight readings are given; in the third column, the heights of instrument; in the fourth column, the foresight readings; and in the fifth column, the elevations of the stations. The sixth column is left for other purposes, such as the description of bench marks, turning points, and important stations. Usually the entire right-hand page is left for remarks.

In the column for stations, the bench marks and turning points are designated *B. M.* and *T. P.*, respectively, and the intermediate points are identified by either letters or numbers. An elevation between two heights of instrument is found by subtracting the corresponding foresight from the *H. I.* preceding it in the notes. It will be observed that the readings and

elevations for bench marks and turning points are given to hundredths of a foot while those on intermediate points are to the nearest tenth. The backsights and foresights must be

LEVEL NOTES

16

Profile for Siding—P. R. R.
Near Greensburg, Pa.
For Location Notes, See Field Book 62, Page 57.

17

Oct. 16, 1925
J. Scott—Leveler
A. Barnes—Rodman

| Sta. | B. S. | H. I. | F. S. | Elev. | | Remarks |
|---|---|---|---|---|---|---|
| B. M. | | | | 100.00 | | Spike on root of white oak |
| | 2.17 | 102.17 | | | | stump 60 feet to left of |
| 0 | | | 4.8 | 97.4 | | Sta. 0. |
| 1 | | | 6.2 | 96.0 | | |
| 2 | | | 9.1 | 93.1 | | |
| 2+50 | | | 8.2 | 94.0 | | |
| 3 | | | 7.6 | 94.6 | | |
| 4 | | | 7.4 | 94.8 | | |
| T. P. | | | 8.54 | 93.63 | | Top of stake near Sta. 4. |
| | 4.58 | 98.21 | | | | |
| 4+60 | | | 11.9 | 86.3 | | Spring Brook. |
| 5 | | | 0.5 | 97.7 | | |
| T. P. | | | 2.67 | 95.54 | | Nail in notch in stump of |
| | 10.32 | 105.86 | | | | beech tree opposite Sta. |
| 5+75 | | | 2.4 | 103.5 | | 5+30. |
| 6 | | | 2.1 | 103.8 | | |
| 7 | | | 6.4 | 99.5 | | |
| 8 | | | 7.7 | 98.2 | | |
| 9 | | | 6.5 | 99.4 | | |
| 10 | | | 8.7 | 97.2 | | |
| T. P. | | | 10.17 | 95.69 | | On rock near Sta. 10. |
| | 2.44 | 98.13 | | | | |
| 11 | | | 2.4 | 95.7 | | |
| 12 | | | 7.2 | 90.9 | | |
| 13 | | | 8.8 | 89.3 | | Spike on root of large |
| B. M. | | | 11.29 | 86.84 | | maple tree 50 ft. to the |
| | | | | | | right of Sta. 13+80. |

FIG. 28

recorded in the field but the heights of instrument and the elevations can be determined at any convenient time.

**60.** In Fig. 29 is shown a modification of the level notes shown in Fig. 28, these notes referring to the same survey.

LEVEL NOTES

| 16 | | | | | | 17 |
|---|---|---|---|---|---|---|
| Profile for Siding—P. R. R.<br>Near Greensburg, Pa.<br>For Location Notes, See Field Book 62, Page 57. | | | | | | Oct. 16, 1925<br>J. Scott—Leveler<br>A. Barnes—Rodman |
| Sta. | B. S. | H. I. | F. S. | I. S. | Elev. | Remarks |
| B. M. | 2.17 | 102.17 | | | 100.00 | Spike on root of white oak |
| 0 | | | | 4.8 | 97.4 | stump 60 ft. to left of |
| 1 | | | | 6.2 | 96.0 | Sta. 0. |
| 2 | | | | 9.1 | 93.1 | |
| 2+50 | | | | 8.2 | 94.0 | |
| 3 | | | | 7.6 | 94.6 | |
| 4 | | | | 7.4 | 94.8 | |
| T. P. | 4.58 | 98.21 | 8.54 | | 93.63 | Top of stake near Sta 4. |
| 4+60 | | | | 11.9 | 86.3 | Spring Brook. |
| 5 | | | | 0.5 | 97.7 | |
| T. P. | 10.32 | 105.86 | 2.67 | | 95.54 | Nail in notch in stump of |
| 5+75 | | | | 2.4 | 103.5 | beech tree opposite Sta. |
| 6 | | | | 2.1 | 103.8 | 5+30 |
| 7 | | | | 6.4 | 99.5 | |
| 8 | | | | 7.7 | 98.2 | |
| 9 | | | | 6.5 | 99.4 | |
| 10 | | | | 8.7 | 97.2 | |
| T. P. | 2.44 | 98.13 | 10.17 | | 95.69 | On rock near Sta. 10. |
| 11 | | | | 2.4 | 95.7 | |
| 12 | | | | 7.2 | 90.9 | |
| 13 | | | | 8.8 | 89.3 | |
| B. M. | | | 11.29 | | 86.84 | Spike on root of large |
| | | | | | | maple tree 50 ft. to |
| | 19.51 | | 32.67 | | | the right of Sta. 13+80. |
| | | | | | | 100.00 |
| | | | | | | 19.51 |
| | | | | | | 119.51 |
| | | | | | | 32.67 |
| | | | | | | 86.84 |

FIG. 29

One difference is that the foresights on turning points and the foresights on intermediate points are kept in separate columns, those on turning points being designated by *F. S.*, and those

on the intermediate points, by *I. S.* By this arrangement the foresights on the turning points can be added conveniently for checking the notes in the manner described in a following article. Another difference from the notes in Fig. 28 is that the backsight and the height of instrument are placed on the same line with the foresight and the elevation for the turning point.

**61.** In Fig. 30 is shown another method of keeping level notes that is in quite general use. The distinguishing feature of this method is a single column for all rod readings, the column being headed Rod. Since the backsights are added and the foresights subtracted, they are indicated by the signs + and −, respectively. The column of rod readings is placed between the columns of heights of instrument and elevations for convenience in performing the operations of addition and subtraction.

These notes refer to the same survey as do the notes in Figs. 28 and 29, but more accurate values are used; the rod readings and elevations for bench marks and turning points are given to thousandths of a foot and the values for intermediate points are taken to the nearest hundredth.

**62. How to Check Level Notes.**—A common method of checking level notes affords a reliable check on the elevations of turning points and heights of instrument, which in a general way is a check on the line of levels as a whole, since all other elevations are deduced from these. The method is based on the fact that all the backsights are additive or + quantities and all the foresights are subtractive or − quantities. Therefore, if the sum of all the backsights in a line of levels, or any portion of it, is added to the elevation of the starting point, and from the sum thus obtained the sum of all the foresights on turning points in the same portion is subtracted, the remainder is the last height of instrument or the elevation of the last turning point, according as the last sight included is a backsight or a foresight.

The application of this method of checking is shown in the notes given in Fig. 29. The elevation of the bench mark near

Station 0 is 100.00 feet. The sum of the backsights, determined by adding the values in the column headed B. S., is

LEVEL NOTES

16 | 17

Profile for Siding—P. R. R.
Near Greensburg, Pa.
For Location Notes, See Field Book 62, Page 57.

Oct. 16, 1925
J. Scott—Leveler
A. Barnes—Rodman

| Sta. | H. I. | Rod | Elev. | | | Remarks |
|---|---|---|---|---|---|---|
| B. M. | | | 100.000 | | | Spike on root of white |
| | 102.172 | + 2.172 | | | | oak stump 60 ft. to |
| 0 | | − 4.75 | 97.42 | | | left of Sta. 0. |
| 1 | | − 6.14 | 96.03 | | | |
| 2 | | − 9.03 | 93.14 | | | |
| 2+50 | | − 8.19 | 93.98 | | | |
| 3 | | − 7.58 | 94.59 | | | |
| 4 | | − 7.36 | 94.81 | | | |
| T. P. | | − 8.543 | 93.629 | | | Top of stake near Sta. 4. |
| | 98.212 | + 4.583 | | | | |
| 4+60 | | −11.93 | 86.28 | | | Spring Brook |
| 5 | | − 0.47 | 97.74 | | | |
| T. P. | | − 2.674 | 95.538 | | | Nail in notch in stump |
| | 105.862 | +10.324 | | | | of beech tree opposite |
| 5+75 | | − 2.39 | 103.47 | | | Sta. 5+30. |
| 6 | | − 2.04 | 103.82 | | | |
| 7 | | − 6.38 | 99.48 | | | |
| 8 | | − 7.07 | 98.19 | | | |
| 9 | | − 6.43 | 99.43 | | | |
| 10 | | − 8.70 | 97.16 | | | |
| T. P. | | −10.171 | 95.691 | | | On rock near Sta. 10. |
| | 98.134 | + 2.443 | | | | |
| 11 | | − 2.45 | 95.68 | | | |
| 12 | | − 7.20 | 90.93 | | | |
| 13 | | − 8.85 | 89.28 | | | |
| B. M. | | −11.292 | 86.842 | | | Spike on root of large |
| | | | | | | maple tree 50 ft. to |
| | | | | | | the right of Sta. 13 |
| | | | | | | +80. |

FIG. 30

19.51 feet; this is added to the elevation of the starting point, the sum being $100.00+19.51=119.51$ feet. The sum of the foresights is 32.67 feet; this is subtracted from the result just

obtained, and the difference, which is $119.51-32.67=86.84$ feet, is the elevation of the bench mark near Station 13+80. When the foresights are added, care must be taken to exclude all readings on intermediate points in case the form of notes is similar to that shown in Fig. 28 or in Fig. 30.

The leveler should check each page of notes, placing a check-mark (√) at the last height of instrument or elevation checked. This should preferably be done as soon as the page is filled, but if there is not time in the field, each day's work should be checked the same night.

## CONDITIONS AFFECTING ACCURACY OF DIRECT LEVELING

### CURVATURE AND REFRACTION

**63. Curvature.**—As has been explained, a level line is a curved line and the line of sight is a horizontal line, tangent to a level line at the instrument. Consequently, the reading

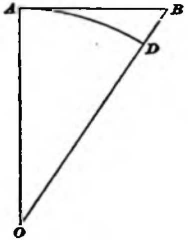


Fig. 31

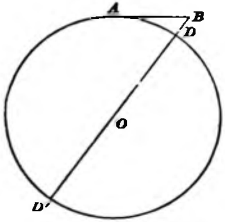


Fig. 32

of a rod held on a point is always greater than the difference in elevation between the horizontal cross-wire and the point because the line of sight cuts the rod at an elevation that is higher than the elevation of the wire. The error in the rod reading due to the curvature of the earth's surface may be computed in the following manner:

In Fig. 31, let $O$ represent the center of the earth, $A\,B$ a horizontal line, and $A\,D$ a level line through the cross-wire of

an instrument at $A$. Then the error due to curvature is $BD$. Since the earth's surface is assumed to be spherical, the level line $AD$ is a circular arc whose radius $OD$ or $OA$ is equal to the earth's radius. In order to show the conditions clearly, the distances in the illustration are shown very much out of proportion; $OA$ is in fact about 20,890,000 feet and for ordinary sights $AB$ is about 400 feet. In geometry it is shown that:

*If from a point without a circle a tangent to the circle and a secant are drawn, the tangent is a mean proportional between the whole secant and its external segment.*

Thus, in Fig. 32, the tangent $AB$ is a mean proportional between the secant $BD'$ and its external segment $BD$, or $\overline{AB}^2 = BD \times BD'$. But $DD'$ is a diameter and is equal to twice the radius $OD$. Hence, in Fig. 32, $\overline{AB}^2 = BD \times (BD + 2\ OD)$. As $BD$ is exceedingly small compared with the diameter of the earth, 2 $OD$, it may be dropped from the quantity within the parenthesis without appreciable error.

Let $e_c$ = error due to curvature, in feet;
$d$ = length of sight, in feet;
$r$ = radius of earth, in feet.

Then, the preceding expression becomes $d^2 = 2\,r\,e_c$, from which

$$e_c = \frac{d^2}{2r}$$

**64. Atmospheric Refraction.**—It is a well-established law of physics that a ray of light in passing from a rarer to a denser medium is bent in a direction toward the denser medium, that is, so that its path will be concave on the side toward the denser medium. This bending of a ray of light is called refraction. Since the atmosphere is densest at the surface of the earth and becomes rarer as the distance from the earth's surface increases, it follows that a ray of light in passing from a higher to a lower elevation by an inclined path will be bent, or refracted, toward the surface of the earth, that is, so that its path will curve in the same general direction as the earth's surface.

Owing to refraction, a ray of light, that apparently is the straight line $BA$ in Fig. 33, actually follows the curved path

$CA$ because $B$ is farther from the earth's surface than $A$; that is, the point observed through the level at $A$ appears to be point $B$ but is really point $C$. Then the error due to atmospheric refraction, which is represented by $BC$ in Fig. 33, is given by the formula

$$e_r = m\frac{d^2}{r}$$

in which $e_r$ = error due to refraction, in feet;
$m$ = numerical coefficient;
$d$ = length of sight, in feet;
$r$ = radius of earth, in feet.

The coefficient $m$ is called the *coefficient of refraction*. Its value varies somewhat for different elevations, but is about .072. The value of $r$ is approximately 20,890,000 feet.

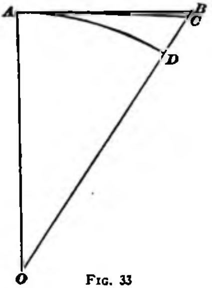

FIG. 33

**65. Combined Error.**—The combined effect of curvature and refraction causes an error represented by the distance $CD$ in Fig. 33, which is evidently equal to the error due to curvature minus that due to refraction. If the combined error in feet is denoted by $e$, then

$$e = e_c - e_r = \frac{d^2}{2r} - m\frac{d^2}{r} = (.5 - m)\frac{d^2}{r}$$

The values of the correction $e$ in decimals of a foot for various values of $d$ expressed in feet are given in Table I; the corrections in this table are computed for $m = .0719$ and $r = 20{,}890{,}000$. For example, if the length of sight is 600 feet, the error due to curvature and refraction is found to be .007 foot. For a length between 300 and 5,280 feet, not listed in the column of distances in Table I, the value of the correction may be found by interpolation. It should be remembered that the correction for curvature and refraction is necessary only in very accurate work and where the difference in lengths of sights for backsights and foresights is great.

TABLE I

CORRECTION FOR THE COMBINED EFFECT OF CURVATURE AND REFRACTION

| d | e | d | e | d | e | d | e |
|---|---|---|---|---|---|---|---|
| 300 | .002 | 1,600 | .052 | 2,900 | .172 | 4,200 | .362 |
| 400 | .003 | 1,700 | .059 | 3,000 | .184 | 4,300 | .379 |
| 500 | .005 | 1,800 | .066 | 3,100 | .197 | 4,400 | .397 |
| 600 | .007 | 1,900 | .074 | 3,200 | .210 | 4,500 | .415 |
| 700 | .010 | 2,000 | .082 | 3,300 | .223 | 4,600 | .434 |
| 800 | .013 | 2,100 | .090 | 3,400 | .237 | 4,700 | .453 |
| 900 | .017 | 2,200 | .099 | 3,500 | .251 | 4,800 | .472 |
| 1,000 | .020 | 2,300 | .108 | 3,600 | .266 | 4,900 | .492 |
| 1,100 | .025 | 2,400 | .118 | 3,700 | .281 | 5,000 | .512 |
| 1,200 | .030 | 2,500 | .128 | 3,800 | .296 | 5,100 | .533 |
| 1,300 | .035 | 2,600 | .139 | 3,900 | .312 | 5,200 | .554 |
| 1,400 | .040 | 2,700 | .149 | 4,000 | .328 | 5,280 | .571 |
| 1,500 | .046 | 2,800 | .161 | 4,100 | .345 | 10,560 | 2.285 |

### ERRORS IN LEVELING

**66. Sources of Error.**—In leveling, the principal sources of error are defects of adjustment of the level, failure of the rodman to hold the rod plumb, and mistakes in reading the rod or recording the readings. The error due to curvature and refraction may become considerable when the difference between the sums of the lengths of the sights for backsights and foresights is great, even though the instrument is in perfect adjustment. If lengths of sights for the backsight and the foresight on turning points from the same position of the level are equal, errors due to imperfect adjustment and also those due to curvature and refraction are balanced, since the error in the foresight is equal to that in the backsight. However, since it is impractical to make the lengths of sights for backsights and foresights exactly equal, it is desirable to avoid errors as far as possible by adjusting the parts of the level by the methods previously explained. The methods of determining when the rod is plumb, also, have been described; and it is apparent that errors due to faulty plumbing can be easily guarded against. Mistakes in determining and recording rod

readings can only be avoided by exercising great care in all operations and by checking whenever possible.

The rays of the sun shining directly on the object glass render the field of view indistinct and the sighting of the telescope uncertain. To prevent this, most instruments are provided with a sunshade, which fits the end of the telescope and projects over the object glass. If the sunshade is lacking, the levelman can hold his hat so as to shade the object glass.

Wind is also a source of error; it sometimes causes the instrument to vibrate, thus preventing the accurate setting of the target and it frequently exerts sufficient pressure against the instrument to cause the bubble to leave the center of its tube. As the pressure is fluctuating, the accurate centering of the bubble is rendered almost impossible. Wind also makes plumbing of the rod more difficult. Under such conditions, the levelman should wait for a lull in the wind, during which, if the rodman is alert, he can usually get a reasonably accurate reading. At a second lull, he can check the value and feel safe regarding it.

Other possible causes of error are the moving of the level due to the settling of the tripod legs in soft or thawing ground or due to their sliding on a smooth surface when a passing train or car produces vibration. It is also important to guard against moving of the turning point or failure of the rodman to hold the rod on the same point for the foresight and the backsight. Sliding of the tripod legs due to vibration can be prevented by proper choice of the set-up and by care in placing the metal points of the legs. Moving of a turning point can be avoided by choosing good permanent points. To avoid holding different points for the backsight and the foresight, turning points should be well defined and well marked when there is a chance of confusion.

**67. Personal Equation.**—There is also what is known as the personal error, sometimes called the personal equation, which is recognized as a defect of vision peculiar to the individual. By reason of this peculiarity of vision, two persons

may observe the reading of a rod, or set the target, on the same sight and under precisely the same conditions, and obtain somewhat different readings. But as this personal equation, or error, is constant for the same person and affects all his observations in the same manner, it does not materially detract from the accuracy of work. Haste is also a fruitful source of error and is little if any aid to progress. Rapid and accurate work can be performed without haste, but work done in a hurry is not usually performed either rapidly or accurately.

**68. Required Degree of Accuracy.**—It has been found from experience that small errors occur much more frequently than large ones, and that those of an accidental character tend to balance each other. A line of levels 20 miles long, in which the rod is read directly to the nearest hundredth of a foot, will give nearly the same results on intermediate points, and nearly the same difference of elevation between the points at its extremities, as a line of precise levels taken with exact target readings. That painful degree of exactness termed hair-splitting is of no advantage in ordinary leveling; it represents very little actual gain in the accuracy of the results, and a very considerable increase in the cost of the work. The degree of accuracy required in each case should be ascertained and the levels taken with sufficient care to obtain that accuracy; greater exactness is an unnecessary waste of time.

If a line of levels is run from any point, completes a circuit, and comes back to the same point, the sum of all the backsights should equal the sum of all the foresights. The difference between these sums is called the *error of closure*. It is a reasonably well-established principle that the total or final value of the accidental errors of direct leveling increases with the length of the circuit and is proportional to the square root of that length. Hence, the permissible error of closure in a line of levels may be expressed by the general formula

$$E = c\sqrt{L}$$

in which $E$ = permissible error of closure;
$c$ = numerical coefficient;
$L$ = length of circuit.

The value of $c$ depends on the character of the survey and on the units in which the values of $E$ and $L$ are expressed. In present practice it is customary to classify leveling according to accuracy, as first-order, second-order, and third-order leveling; first-order leveling has the highest degree of accuracy. The permissible errors of closure for these orders are as follows:

For first-order leveling,

$$E = .017\sqrt{M}$$

For second-order leveling,

$$E = .035\sqrt{M}$$

For third-order leveling,

$$E = .05\sqrt{M}$$

In these formulas, $E$ denotes the permissible error of closure, in feet, in a leveling circuit; and $M$ denotes the length of the circuit, in miles. For example, in the case of a leveling circuit that is 100 miles long, the permissible error of closure is $.017\sqrt{100} = .17$ foot for first-order leveling; $.035\sqrt{100} = .35$ foot for second-order leveling; and $.05\sqrt{100} = .5$ foot for third-order leveling.

**69. Check-Levels.**—Before the elevations obtained by a line of levels which does not form a closed circuit are finally adopted as a basis for construction work, their accuracy should be verified by another line of levels over the most important and permanent points whose elevations were taken by the former line. This second line then completes the circuit. Levels for the purpose of verifying previous work are called *check-levels*, or *test levels*. In running check-levels, the most common practice is to take only the elevations of bench marks and important permanent points with such turning points as may be necessary in order to cover the distance; nearly all the intermediate points are omitted. The check-levels should always be run in the direction opposite to that of the original line in order to eliminate the effect of unavoidable errors. If the variation in the elevation of a point as given by the two sets of levels is less than the permissible value of $E$, the true elevation of the point is determined as follows: The

difference in elevation between the starting point and the point in question is computed from each line of levels independently. Then, the average difference is added to, or subtracted from, the elevation of the starting point. But if the variation is greater than the allowable value of $E$, the levels should be run again over that portion of the line in which the variation occurs, in order to determine which of the elevations is correct. Where the true elevation of a bench mark is found to differ from the elevation marked on it, the value should be corrected.

### EXAMPLES FOR PRACTICE

1. In work that may be classified as third-order leveling, what is the permissible error of closure in a line of levels 18 miles long? Ans. .212 ft.

2. In work of the United States Geological Survey of first-order accuracy, what is the permissible error of closure in a line of levels 50 miles long? Ans. .120 ft.

3. In a survey of second-order accuracy, what is the permissible error of closure in running a line of levels having a length of 30 miles? Ans. .192 ft.

### PROFILES

**70. Definition.**—A *profile* is a representation of the vertical section along a survey line. It shows the horizontal and vertical distances between points on the line as if the line were straight; that is, in a vertical plane. The actual line may be partly curved as in the case of a railroad or highway, or broken as in a sewer or water line.

The vertical distances on a profile are usually represented to a larger scale than the horizontal distances in order to make more distinct the irregularities of the surface along the line of survey. Therefore, a profile is seldom a true section along the line, except in geological work, where the horizontal and vertical distances are represented to the same scale. For railroad work, profiles are commonly constructed to a horizontal scale of 400 feet to the inch and a vertical scale of 20 feet to the inch. For municipal work, it is common to use a horizontal scale of 40 feet to the inch and a vertical scale of

4 feet to the inch. Other scales are also used according to the character and requirements of the work.

**71. Construction.**—A profile can be constructed on plain paper in the following manner:

A reference line is first drawn near the lower edge of the paper. It is usually convenient to make the elevation of the reference line slightly less than that of the lowest point on the survey, because all points to be located will then be above this line. On the reference line are laid off to the chosen horizontal scale the distances from the starting point of the survey to the stations whose elevations have been taken, and the station points thus located are numbered at intervals for identification. The bench marks and turning points are not laid off along the reference line, because usually they are neither on the surface of the ground nor along the survey line for which a profile is desired. A vertical line of indefinite length is then drawn upwards from each station point and a point representing the ground surface is located on each vertical line by laying off to the vertical scale the difference in elevation between the reference line and the ground surface at the station. These points are finally connected by a heavy freehand line, which constitutes the profile of the survey line. The profile line is drawn freehand to correspond more nearly to the irregularities of the earth's surface.

In Fig. 34 is shown the profile that would be drawn from the level notes given in Fig. 28. The horizontal scale is 1 inch equals 400 feet, and the vertical scale is 1 inch equals 20 feet. The elevation of the reference line is taken as 70 feet, this number being marked at each end of the line; on long lines the elevation would be marked at several intermediate points. The station numbers along the reference line in Fig. 34 are numbered every 500 feet, though often they are numbered at every 100 feet or at every 1,000 feet. The calculations for drawing this profile are as follows:

The elevation of Station 0, as given in Fig. 28, is 97.4; therefore, its height above the reference line is $97.4-70=27.4$ feet. This distance laid off on the vertical line at Station 0

locates the position of the surface at that place. The elevation of Station 1 is 96.0; therefore, a distance of $96.0-70=26.0$ feet is laid off on the vertical line at Station 1, thus locating the surface at that point. To locate the surface at Station 2, the distance to be laid off on the vertical line at that station is $93.1-70.0=23.1$ feet. The heights of the remaining stations

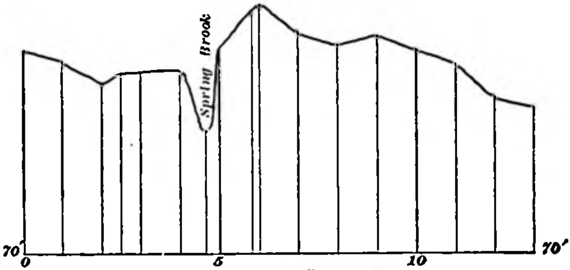


FIG. 34

above the reference line are laid off in a similar manner on the vertical lines at the corresponding stations. Finally, the surface line is drawn between the points as already described.

**72. Profile Paper.**—In order to facilitate the construction of profiles, paper prepared especially for the purpose is commonly used; this has horizontal and vertical lines in pale green, blue, or orange, so spaced as to represent certain distances to the horizontal and vertical scales. Such paper is called profile paper. The most common form of profile paper is divided into $\frac{1}{4}$-inch squares. Then the space between each two horizontal lines is divided into 5 equal parts by lighter horizontal lines, the distance between these light lines, therefore, being $\frac{1}{20}$ inch. Consequently, with the scales commonly used for railroad profiles, the light horizontal lines are 1 foot apart and the vertical lines are 100 feet apart. Of course, any desired values can be assumed for the spaces according to the requirements of the work. In order to aid in estimating distances,

each tenth line in both directions is made extra heavy. A piece of profile paper showing the profile for the level notes given in Fig. 28 is illustrated in Fig. 35. The method of locating the points is similar to that explained in the preceding article but the distances can be found from the lines on the paper without actually measuring them. The elevation of the extra-heavy line is assumed to be 100 feet, and each division between light horizontal lines represents 1 foot. Then the ground at Station 0 is located 2.6 spaces below the extra-heavy line, and the ground at Station 1 is 4 spaces below. At Station 4+60, the elevation is 86.3, and the ground is 13.7 spaces

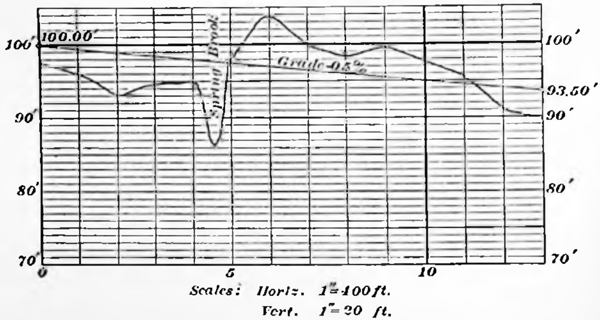


Fig. 35

below the extra-heavy line; since each heavy line represents 5 spaces, the point can be located by counting 2 heavy lines and then 3.7 spaces additional. The ground at Station 4+60 can also be located by first determining the heavy line at elevation 85.0 and then proceeding upwards 1.3 spaces.

**73. Grade Lines.**—In important engineering work, before the actual construction is begun, it is customary to decide on the position of some prominent line in the completed work and to adopt it as a line of reference; this line is called a *grade line.* The elevation of the grade line at any point is known as the *grade* at that point. When planning a railroad, the grade line

represents the proposed position of the base of the rail; and for a street or highway, the grade line is the finished surface at the center line. In constructing a railroad or a highway, however, the proposed surface of the ground, which is called the *subgrade*, is taken as the grade line. Grade lines are usually straight for considerable distances. The grade line is shown with the profile of the present ground so that the distance of the present ground above or below grade can be readily seen. If a point on the profile of the original ground is above the subgrade, material must be excavated; while if a point on the profile of the ground is below the subgrade, it is necessary to fill in material. The process of excavating and filling to make the surface of the ground correspond to the proposed subgrade is called *grading*.

The principal purpose for which a profile is constructed is to enable the engineer to establish the grade line. For a railroad or a highway, the subgrade should, when possible, be located in such a position that the excavation and embankment will be nearly the same in amount. The position of the grade line having been determined, it is drawn on the profile in red ink.

**74. Rate of Grade.**—The inclination of a grade line to the horizontal may be expressed by the relation between the rise or fall of the line and the corresponding horizontal distance. The amount by which the grade line rises or falls in a unit horizontal distance is called the *rate of grade* or the *gradient*. The rate of grade is usually expressed as a percentage; that is, as the rise or fall in a horizontal distance of 100 feet. For instance, if the grade line rises 2 feet in 100 feet, it has an *ascending grade of 2 per cent.*, which is written $+2\%$. If the grade line falls 1.83 feet in 100 feet, it has a *descending grade of 1.83 per cent.*, which is written $-1.83\%$. The sign $+$ indicates a rising grade line and the sign $-$ indicates a falling grade line. Sometimes the sign $\%$ is omitted.

The rate of grade is written along the grade line on the profile, and the elevation of grade is written at the extremities of the line and at each point where the rate of grade changes.

It is common practice to enclose in small circles the points on the profile where the rate of grade changes.

**75.** All computations relating to rates of grade can be performed by applying the following rules:

**Rule I.**—*The rate of grade, in per cent., is equal to the total rise or fall in any horizontal distance divided by the horizontal distance and multiplied by 100.*

**Rule II.**—*The total rise or fall of a grade line in any given horizontal distance is equal to the rate of grade, in per cent., multiplied by the horizontal distance and divided by 100.*

**Rule III.**—*The horizontal distance in which a given grade line will rise or fall a certain amount is equal to the amount of rise or fall divided by the rate of grade, in per cent., and multiplied by 100.*

EXAMPLE.—The total rise of a certain grade line is 66 feet in a horizontal distance of 1 mile. (*a*) What is the rate of grade? (*b*) If the elevation of the grade at Station 2+00 is 150.00 feet, what is the elevation of the grade at Station 13+00?

SOLUTION.—(*a*) In 1 mile there are 5,280 feet; then, by rule I, the rate of grade is

$$\frac{66\times 100}{5{,}280}=1.25 \text{ per cent.} \quad \text{Ans.}$$

(*b*) The horizontal distance between Station 2+00 and Station 13+00 is 1,100 feet. The total rise of the grade line between these two stations is, according to rule II,

$$\frac{1{,}100\times 1.25}{100}=13.75 \text{ ft.}$$

The elevation of the grade at Station 2 is 150.00 feet; therefore, the elevation of the grade at Station 13 is 150.00+13.75=163.75 ft. Ans.

**76. Cut and Fill.**—The subject of leveling does not properly include cut and fill, but since grading is associated very closely with the subjects of profiles and grade lines, the following explanation is given here. The vertical distance of the subgrade below the surface line at any point, as shown on the profile, will be the depth of excavation, or cutting, necessary to bring the surface of the ground to the established grade at that point. Likewise the vertical distance of the subgrade

above the surface line at any point will represent the depth of embankment, or filling, necessary to bring the surface of the ground to the proposed grade. The depth of excavation and

GRADING NOTES

| Sta. | H. I. | Rod | Elev. of Surf. | Elev. of Grade | Cut or Fill | Remarks |
|---|---|---|---|---|---|---|
| B. M. | | | 100.000 | | | Spike on root |
| | 102.172 | + 2.172 | | | | of white oak |
| 0 | | − 4.75 | 97.42 | 100.00 | F. 2.58 | stump 60 ft. to |
| 1 | | − 6.14 | 96.03 | 99.50 | F. 3.47 | left of Sta. 0. |
| 2 | | − 9.03 | 93.14 | 99.00 | F. 5.86 | |
| 2+50 | | − 8.19 | 93.98 | 98.75 | F. 4.77 | |
| 3 | | − 7.58 | 94.59 | 98.50 | F. 3.91 | |
| 4 | | − 7.36 | 94.81 | 98.00 | F. 3.19 | |
| T. P. | | − 8.543 | 93.629 | | | Top of stake near |
| | 98.212 | + 4.583 | | | | Sta. 4. |
| 4+60 | | −11.93 | 86.28 | 97.70 | F. 11.42 | Spring Brook. |
| 5 | | − 0.47 | 97.74 | 97.50 | C. 0.24 | |
| T. P. | | − 2.674 | 95.538 | | | Nail in notch in |
| | 105.862 | +10.324 | | | | stump of beech |
| 5+75 | | − 2.39 | 103.47 | 97.12 | C. 6.35 | tree opposite |
| 6 | | − 2.04 | 103.82 | 97.00 | C. 6.82 | Sta. 5+30. |
| 7 | | − 6.38 | 99.48 | 96.50 | C. 2.98 | |
| 8 | | − 7.67 | 98.19 | 96.00 | C. 2.19 | |
| 9 | | − 6.43 | 99.43 | 95.50 | C. 3.93 | |
| 10 | | − 8.70 | 97.16 | 95.00 | C. 2.16 | |
| T. P. | | −10.171 | 95.691 | | | On rock near |
| | 98.134 | + 2.443 | | | | Sta. 10. |
| 11 | | − 2.45 | 95.68 | 94.50 | C. 1.18 | |
| 12 | | − 7.20 | 90.93 | 94.00 | F. 3.07 | Spike on root |
| 13 | | − 8.85 | 89.28 | 93.50 | F. 4.22 | of large maple |
| T. P. | | −11.292 | 86.842 | | | tree 50 ft. to |
| | | | | | | the right of |
| | | | | | | Sta. 13+80. |

FIG. 36

the depth of embankment are, for short, commonly spoken of as the *cut* and the *fill*, respectively.

The cut or the fill is usually calculated for each station in connection with the level notes and recorded in the notebook.

At each station where the elevation of the surface exceeds the elevation of the grade, the difference will be a cut. At each station where the elevation of the grade exceeds the elevation of the surface, the difference will be a fill. The cuts are designated either by the letter *C.* or by the sign +, and the fills are indicated either by *F.* or by −.

**77. Calculation of Cut and Fill.**—The grading notes given in Fig. 36 are a repetition of the typical level notes of Fig. 30, to which are added the elevation of grade and the cut or the fill at each station. This form of notes is convenient since the six columns on the left-hand page of the notebook are sufficient.

The elevation of grade at Station 0 is established at 100.0 feet, and the rate of grade is taken as −0.5. Hence, the elevation of grade at Station 1 is $100.0-.5=99.5$ feet; the elevation of grade at Station 2 is $99.5-.5=99.0$ feet. The elevation of grade at each succeeding station is determined in a similar manner and written in the column headed Elev. of Grade. The elevation of grade at Station 4+60 is $98.0-\frac{.5\times 60}{100}=97.7$ feet, and the elevation of grade at Station 5+75 is $97.5-\frac{.5\times 75}{100}$ $=97.12$ feet.

The difference between the elevation of grade and the surface elevation at Station 0 is $100.00-97.42=2.58$ feet; since the elevation of grade exceeds the elevation of the present ground, a fill is necessary as indicated in the notes. At Station 5, the difference between the present surface and the proposed grade is $97.74-97.50=0.24$ feet; since the present surface is higher, material must be cut.

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#### EXAMPLES FOR PRACTICE

1. Between Stations 10+00 and 25+00 of a certain survey there is a grade of +2.00 per cent. If the elevation of the grade at Station 10+00 is 48.00 feet, what is the elevation of the grade (*a*) at Station 15+00, (*b*) at Station 18+75, and (*c*) at Station 23+67?

Ans. (*a*) 58.00 ft. (*b*) 65.50 ft. (*c*) 75.34 ft.

2. If the elevation of the grade at Station 0 is 150.10 feet and that of the grade at Station 15+80 is 67.15 feet, what is the rate of grade? Ans. −5.25%

3. What is the total rise of a +3.75 per cent. grade in a distance of 2,640 feet? Ans. 99.00 ft.

4. In what horizontal distance will a grade of +4.2 per cent. effect a rise of 94.50 feet? Ans. 2,250 ft.

## BAROMETRIC LEVELING

### THE BAROMETER

**78. Air Pressure.**—A body submerged in water is subjected to a pressure caused by the weight of the water above it; and the deeper the body is below the surface of the water, the greater is the pressure. The air surrounding the earth, known as the *atmosphere*, also has weight and, therefore, exerts a pressure on bodies, which is called *atmospheric pressure*. This pressure depends on the distance below the surface of the atmosphere, or on the distance above or below sea level, which is the usual reference surface. The atmospheric pressure is greater at sea level than at higher points, and decreases as the altitude increases. The difference between the atmospheric pressure at two points may, therefore, serve as a measure of the difference in elevation between the points.

**79. Barometers.**—Instruments for measuring atmospheric pressure are known as barometers. Barometers are of two general kinds. In one kind, called the *mercurial barometer*, the atmospheric pressure is indicated by the height of a column of mercury. In the other kind, called the *aneroid barometer*, the pressure is indicated by the deflection of the sides of a metal box from which the air has been removed.

Mercurial barometers are more accurate than aneroid barometers, but, as they are not so portable, their use is confined almost exclusively to laboratory work. Aneroid barometers are now made sufficiently accurate for ordinary purposes; and the ease with which they can be carried makes them well suited for leveling where only approximate elevations are required.

**80. Mercurial Barometer.**—A mercurial barometer, illustrated in Fig. 37, is constructed as follows: A glass tube, about 36 inches long and closed at one end, is filled with mercury. Then, while the open end of the tube is covered to prevent the escape of the mercury, the tube is inverted and the open end is submerged in mercury contained in a cup. When the open end of the tube is uncovered, and the mercury allowed to come to rest, the surface of the mercury in the tube will be about 30 inches vertically above that in the cup.

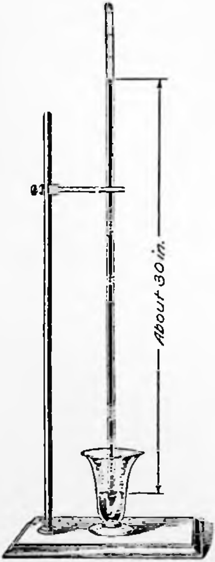

FIG. 37

The reason why the surface of the mercury does not have the same elevation in the tube and in the cup is that there is no air above the mercury in the tube, but the mercury in the cup is exposed to the atmospheric pressure. Therefore, the atmospheric pressure balances a column of mercury about 30 inches high, the weight of which can be determined. The weight of a cubic inch of mercury is .49 pound; and if the tube is assumed to have an area of 1 square inch, the volume of the mercury column is $30 \times 1 = 30$ cubic inches, and its weight is $.49 \times 30 = 14.7$ pounds. Since the weight acts on an area of 1 square inch, the pressure due to this weight is 14.7 pounds per square inch. This pressure is balanced by the atmospheric pressure; hence, the atmospheric pressure at sea level for a temperature of 60° Fahrenheit is 14.7 pounds per square inch. Such a pressure is often called a pressure of 1 atmosphere and is the standard commonly used for the atmospheric pressure when no other is given. It must be remembered that the atmospheric pressure is affected by other conditions besides altitude, and that the barometer reading fluctuates as the weather changes.

The temperature of the air also affects the pressure somewhat, cold air being heavier than hot air. The reading of a barometer, therefore, depends on the temperature as well as on the altitude. For this reason the temperature should be observed whenever the pressure is read.

**81.** Atmospheric pressure is often expressed by the height of a mercury column which it supports. For instance, if the mercury in the tube of a barometer is 29 inches above the mercury in the cup, the pressure is said to be 29 inches; similarly, a pressure of 31 inches means that the mercury in the tube of a barometer is 31 inches above the mercury in the cup. When the atmospheric pressure varies, the height of the mercury column changes; an increase in pressure causes the surface of the mercury in the tube to rise, and a decrease in pressure allows the surface to drop. The higher the altitude, the lower is the pressure, and, consequently, the shorter is the column of mercury. The atmospheric pressure at sea level is about 30.0 inches; at an elevation of 1,000 feet above sea level, it is about 28.9 inches; at 2,000 feet, 27.8 inches.

**82. Aneroid Barometer.**—There are many types of aneroid barometers, but all are essentially the same in action. An aneroid barometer consists of a metal case with a glass face; and in the case is a flat cylindrical chamber from which the air has been exhausted. The circular sides of this chamber, corresponding to the ends of a cylinder, consist of thin corrugated metal plates reinforced at the center by strong metal disks. The variation in the atmospheric pressure is indicated by changes in the deflection of the plate. As the atmospheric pressure increases, the difference between the pressures inside and outside the chamber becomes greater; consequently, the plate is deflected more. Conversely, as the atmospheric pressure decreases, the difference between the pressures outside and inside the chamber becomes less, and the elastic resistance of the metal reduces the deflection. The motion of the plate, greatly multiplied, is transmitted by means of a system of levers to a pointer that moves over graduated scales; the pointer and scales are visible through the glass face of the

instrument, as shown in Fig. 38. Most aneroid barometers have two scales as shown in the figure. The inner, called the *mercury scale*, indicates the pressure in inches of mercury at the time of observation; readings on the outer, or *altitude scale*, may be used for determining differences in elevation directly.

**83. Altitude and Mercury Scales.**—Some aneroid barometers, like that in Fig. 38, show altitudes above sea level


FIG. 38

only; others read both above and below sea level. Also the limits of the altitudes vary considerably. The barometer shown in Fig. 38 measures up to 10,000 feet above sea level, but similar instruments are made that measure to 20,000 feet.

There are various methods of graduating the scales. On the mercury scale in Fig. 38, the inches are numbered with

large figures, and every alternate tenth of an inch is numbered with a smaller figure. Each tenth of an inch is divided into 5 parts, and, therefore, each very small division represents $\frac{1}{5} \times .1 = .02$ inch. In case the pointer is between graduations, the nearest half-division, that is, the nearest hundredth of an inch, can be easily estimated. For the position shown, the pointer reads about 29.43 inches.

On the altitude scale in Fig. 38, each 1,000 feet is numbered and the smaller figures indicate the hundreds. Each 100 feet is divided into 5 parts and each small division, therefore, is $\frac{1}{5} \times 100 = 20$ feet. By means of the vernier, which is shown near the 1,400-foot graduation, the altitude can be read to the nearest foot.

The vernier can be set at any point on the scale by turning the milled screw shown at the top of the case. To assist in reading the scale and vernier, a movable magnifying glass, shown at the 3,000-foot graduation in Fig. 38, is provided.

**84.** The mercury scale on a barometer records accurately the atmospheric pressure, but the pressure at any altitude varies for different locations, and at the same place it is continually changing as the temperature changes. Furthermore, as shown in Fig. 38, the zero of the altitude scale generally coincides with 31 inches on the mercury scale, which is seldom the value of the atmospheric pressure at sea level. The elevation indicated on the altitude scale of a barometer may, therefore, differ considerably from the true value at the point of observation. However, if observations are taken at two points, the difference in elevation between the points can be determined with sufficient accuracy for ordinary purposes.

Barometers are made in which the zero of the altitude scale can be moved to coincide with any graduation on the mercury scale. But, for good work, zero of the altitude scale should be kept at the mercury reading that was assumed to correspond to sea level in graduating the altitude scale.

**85. Compensation for Temperature.**—Unless a barometer is specially designed, variation in temperature so affects its parts that different readings are obtained for the same actual

pressure. Therefore, some instruments are so constructed that this effect of temperature changes is automatically corrected. Such barometers are marked, "Compensated," as shown in Fig. 38.

**86. Care of Aneroid Barometer.**—An aneroid barometer is necessarily of delicate construction. Therefore, great care should be exercised, when the instrument is being used and carried, to protect it from shocks and jars, from moisture, from the direct rays of the sun, and from the heat of the body or an artificially warmed room. Each aneroid barometer is provided with a leather case in which it should always be carried.

## DETERMINATION OF ELEVATIONS WITH A BAROMETER

### INTRODUCTION

**87. Variations in Pressure.**—If the pressure for a given altitude were constant, a certain barometer reading would always indicate a certain altitude. Unfortunately, however, the pressure of the air for a given elevation varies. The three principal factors in the variation are time, location, and temperature.

The pressure of the air changes, as the weather changes, from day to day and from hour to hour; in dry settled weather it is about from 30.5 to 31 inches of mercury at sea level, while in stormy weather it is about 1.5 inches lower. It is also a matter of observation that at the same time the weather may be different in different places; consequently, the atmospheric pressure depends on the locality. Then, too, a given volume of cool air weighs more than an equal amount of warm air; hence, the pressure of the air varies with the temperature.

**88. Taking Readings.**—When a barometer reading is taken, the instrument should have as nearly as possible the temperature of the surrounding air, which temperature should be observed at the same time. A mercurial barometer should be kept vertical and the surface of the mercury in the cup should

be at the elevation of the point the height of which is to be determined. An aneroid barometer should be horizontal and in the open air. The case should be tapped gently before the reading is taken.

**89. Formulas for Difference in Elevation.**—Since the pressure of the air varies with the altitude, it follows that the difference in elevation between two points is measured by the difference between the pressures at the points. It has also been found that the pressure of the air is affected by its temperature. When the pressures and temperatures at two points are known, the approximate difference in elevation between the points may be found by one of the following formulas. These formulas are derived from the data of many experiments, and the results are sufficiently accurate for practical use.

When the pressures in inches of mercury are observed, the formula for the difference in elevation is

$$z=58.4\,\frac{p_1-p_2}{p_1+p_2}(836+t_1+t_2) \qquad (1)$$

in which $z$ = difference in elevation between two stations, in feet;

$p_1$ = pressure at lower station, in inches of mercury;

$p_2$ = pressure at higher station, in inches of mercury;

$t_1$ = temperature at lower station, in degrees Fahrenheit.

$t_2$ = temperature at higher station, in degrees Fahrenheit;

When the altitude scale on the barometer is read, the difference in elevation may be found by the formula

$$z=(h_2-h_1)\frac{900+t_1+t_2}{1{,}000} \qquad (2)$$

in which $z$, $t_1$, and $t_2$ have the same meanings as in formula **1**;

$h_2$ = altitude reading at upper station, in feet;

$h_1$ = altitude reading at lower station, in feet.

Since the results obtained by the use of these formulas are only approximate, they may be taken to the nearest 10 feet.

EXAMPLE 1.—Suppose the barometer at one station reads 26.25 inches with the temperature at 72° F., and at a second station it reads 24.95 inches with the temperature at 46° F. What is the difference in elevation?

SOLUTION.—In formula **1**, substitute the values $p_1 = 26.25$ in., $p_2 = 24.95$ in., $t_1 = 72°$, and $t_2 = 46°$. Then,

$$z = 58.4 \times \frac{26.25 - 24.95}{26.25 + 24.95}(836 + 72 + 46) = 1{,}410 \text{ ft.} \quad \text{Ans.}$$

EXAMPLE 2.—The reading on the altitude scale at a station is 437 feet and the temperature is 60° F. At another station, the barometer reading is 1,118 feet and the temperature 50° F. Find the difference in elevation between the stations.

SOLUTION.—In formula **2**, substitute the values $h_1 = 437$ ft., $h_2 = 1{,}118$ ft., $t_1 = 60°$, and $t_2 = 50°$. Then,

$$z = (1{,}118 - 437)\frac{900 + 60 + 50}{1{,}000} = 690 \text{ ft.} \quad \text{Ans.}$$

### EXAMPLES FOR PRACTICE

1. The reading of a barometer at a station is 28.44 inches and the temperature is 60°. At another station the barometer reading is 24.33 inches and the temperature is 40°. Calculate the difference in elevation between the two stations. Ans. 4,260 ft.

2. The readings of the altitude scale at two stations are 4,526 feet and 5,910 feet; and the respective temperatures are 42° and 34°. What is the difference in elevation? Ans. 1,350 ft.

3. The barometer readings at two points are 2,350 feet and 6,600 feet. If the temperatures are, respectively, 58° and 42°, what is the difference in elevation? Ans. 4,250 ft.

### FIELD OBSERVATIONS

**90. Method With One Barometer.**—The field observations in barometric leveling may be made with either one barometer or two. If one instrument is used, it is placed first at the point of known elevation and then is moved as rapidly as possible to the other stations whose elevations are required. At each point, the time, the temperature, and the barometer reading are observed. The difference in elevation between the starting point and each other station can be computed by one of the formulas of Art. **89**. The elevation of any station is then found from the elevation of the reference point by adding or subtracting the difference in elevation for the station in question.

In the preceding method large errors are likely to be introduced on account of the time which elapses between observations at the different points. Better results are obtained if the observations are repeated, the points being visited in the reverse order, and the average of the two values for each station taken.

**91. Method With Two Barometers.**—The best way to determine differences of elevation from air pressures is to use two barometers. One instrument is kept at a point of known elevation. At intervals, the time, the temperature, and the barometer reading are observed. The other barometer is moved from station to station, and at each, the time, the temperature, and the barometer reading are recorded. The same scale must be read on both barometers. The data for the reference point for any time can be obtained from the observed values by interpolation, the variation in conditions being assumed to be uniform between observations.

| Observations at Station of Known Elevation | | | Observations with Moving Barometer | | | |
|---|---|---|---|---|---|---|
| Elev. of $a=604$ | | | | | | |
| *Time A. M.* | *Barom. In.* | *Temp. Deg. F.* | *Sta.* | *Time A. M.* | *Barom. In.* | *Temp. Deg. F.* |
| 8:30 | 29.20 | 63° | | | | |
| 9:00 | 29.36 | 65° | *b* | 9:00 | 28.65 | 60° |
| 9:30 | 29.55 | 68° | | | | |
| 10:00 | 29.65 | 72° | *c* | 9:55 | 30.00 | 58° |
| 10:30 | 29.68 | 75° | *d* | 10:22 | 28.06 | 54° |
| 11:00 | 29.62 | 77° | *e* | 10:55 | 29.80 | 50° |

Fig. 39

Typical field notes for leveling with two barometers are shown in Fig. 39.

The elevations at the stations are determined in the following manner. Since observations at *a* and *b* were taken at the same time, the difference in elevation is found simply by applying formula 1 of Art. 89. Thus the vertical distance between *a* and *b* is $z=58.4\times\frac{29.36-28.65}{29.36+28.65}(836+65+60)=690$ feet.

Station $b$ is higher than $a$ because the barometer reading at $b$ is lower. The elevation of $b$ is, therefore, $604+690=1{,}294$ feet.

In order to find the elevation at $c$, it is necessary to determine the barometer reading and the temperature at $a$ at 9:55 A.M. The data for the reference point $a$ at any time can be obtained from the observed values by interpolation, as the variation in conditions is assumed to be uniform between observations. The difference in time between 9:30 and 10:00 is 30 minutes; the change in pressure at $a$ for this interval is $29.65-29.55=.10$ inch and the change in temperature is $72°-68°=4°$. The interval between 9:30 and 9:55 is 25 minutes. The variation in pressure at $a$ from 9:30 to 9:55, therefore, is $\frac{25}{30}\times.10=.08$ in., and the variation in temperature is $\frac{25}{30}\times4=3°$. The values of the pressure and temperature at $a$ at 9:55 equal, respectively, $29.55+.08=29.63$ in. and $68+3=71°$. Then the difference in elevation between $a$ and $c$ is $z=58.4\times\frac{30.00-29.63}{30.00+29.63}(836+58+71)=350$ feet, and since $c$ is lower than $a$ the elevation of $c$ is $604-350=254$ feet. The elevations of $d$ and $e$ are obtained in a similar manner.

**92. Accuracy of Barometric Leveling.**—The differences between the altitudes as determined by direct leveling and those computed from barometer readings are often considerable. It is impossible to give any precise rule. First, because of the inaccuracy of the barometer itself, an error of 5 or 10 feet may be expected. Then the results depend on many varying quantities, such as the weather, the vertical and horizontal distances between the stations, the method of conducting the work, and the care of the observer. As an indication of the value of barometric leveling, it may be stated that a variation of 20 feet may be expected; and when the points are far apart, one of 50 feet is possible even in good work.

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**EXAMPLES FOR PRACTICE**

1. From the notes in Fig. 39, determine the elevation of Station $d$.
Ans. 2,174 ft.

2. Find the elevation of Station $e$. Ans. 444 ft.

# COMPASS SURVEYING

Serial 3069-2 Edition 1

## THE COMPASS

### PRELIMINARY EXPLANATIONS

#### INTRODUCTION

**1. Compass and Transit.**—The main instruments used for measuring angles in surveying are the *compass* and the *transit*. Formerly, the compass was used extensively in surveying, but it has been largely superseded by the transit, which is a more accurate instrument and is more suitable for most kinds of surveying work. At present, the compass is used chiefly for relocating lines of old surveys. It is also employed to a limited extent in preliminary work on railroads and occasionally in new land surveys where the property is of little value.

**2.** One important feature that distinguishes the method of measuring angles with the compass from that with the transit is that by means of a transit the angle formed by any two lines can be measured directly, whereas the compass can serve only for measuring the angle that a given line makes with the magnetic meridian or magnetic needle. To determine the angle between two lines by means of the compass, the angle that each line makes with the magnetic needle must first be measured; and from these values, the required angle may then be calculated as follows:

Let $AB$, Fig. 1, be the direction of the magnetic needle. To determine the angle $CAD$ by means of the compass, the angles

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$BAC$ and $BAD$ that $AC$ and $AD$ make with the magnetic needle must first be measured. Then in the case shown in (*a*),

$$\text{angle } CAD = \text{angle } BAC + \text{angle } BAD$$

and in the case shown in (*b*),

$$\text{angle } CAD = \text{angle } BAD - \text{angle } BAC$$

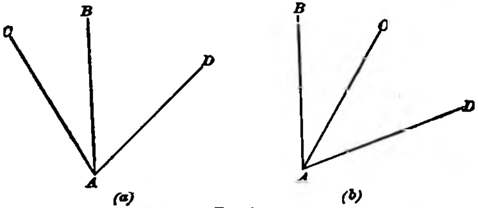


FIG. 1

3. **Meridians.**—The directions of the lines of a survey are usually given with respect to some fixed reference line, which is called a *meridian*. At each point on the earth's surface, there are two definite lines, the *true meridian* and the *magnetic meridian*, which are generally used in surveying.

4. **True Meridian.**—The axis on which the earth rotates is an imaginary line cutting the earth's surface in two points known as the *north geographic pole* and the *south geographic pole*. The line passing through a point on the earth's surface and directed toward the geographic poles is called the *true meridian* at the given point.

5. **Magnetic Meridian.**—If a magnetized steel bar is allowed to rotate on a pivot near its center, the bar will always take very nearly the same direction at any place. The line thus indicated is called the *magnetic meridian* at the given point; it has the general direction of the true meridian. A magnetized bar is acted upon by two magnetic forces, which are assumed to be at two points, called *magnetic poles*, on opposite sides of the earth's surface and near the ends of the earth's axis. The magnetic pole near the north geographic pole is the *north magnetic pole*, and the other, which is near the south geographic pole, is the *south magnetic pole*. The same end of a

freely-suspended magnetized bar will always point toward the north and this end is known as the *north end* of the bar; the other end of the bar is its *south end*.

Meridians are circles on the earth's surface and meet at the poles. But, for the purposes of ordinary surveying where relatively small areas are considered, meridians are treated as parallel straight lines that lie in a horizontal plane.

**6. Azimuths.**—The *azimuth* of a line is the angle between a meridian and the line, and is always measured from the meridian in a clockwise direction from 0° to 360°: in surveying, azimuths are generally measured from the north point, but sometimes the south point is used. Unless otherwise stated, azimuths will be considered as measured from the north. In Fig. 2, $NS$ represents a meridian with $N$ toward the north; the azimuth of $OA$ is 115°, that of $OB$ is 246°, and that of $OC$ is 300°. Azimuths are called *true azimuths* when measured from a true meridian, and *magnetic azimuths* when measured from a magnetic meridian.

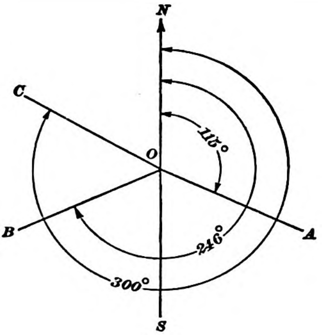

FIG. 2

**7. Bearings.**—The angle between a meridian and a line may be given by the *bearing* of the line instead of by the azimuth. In determining bearings, the plane around a point is divided into four quadrants by two lines, of which one is in the direction of the meridian and the other is at right angles to the meridian. Bearings are reckoned from 0° to 90° in each quadrant, the zero points being in the meridian and the 90° points in the east-and-west line. It is thus seen that there are four lines, one in each quadrant, which make the same angle with the meridian. In order to distinguish between

these lines, the letters $N$, $E$, $S$, and $W$, indicating north, east, south, and west, respectively, are used to show the quadrant. Thus, if a line is in the northeast quadrant, its bearing is written with the letter $N$ preceding the value of the angle, and the letter $E$ following the angle. For a line in the southeast quadrant, the letter $S$ precedes the angle and $E$ follows. Lines in the other quadrants are indicated by the corresponding letters in a similar manner. For example, the bearing of the line $OP_1$ in Fig. 3 is written N 60° E, called *north 60° east*, because the line $OP_1$ lies in the quadrant between north and east and the angle with the meridian is 60°. The bearing of the line $OP_2$ is N 42° W, called *north 42° west*, because $OP_2$ is in the quadrant between north and west and the angle with the meridian is 42°. Similarly the bearing of the line $OP_3$ is S 70° W (south 70° west) and the bearing of the line $OP_4$ is S 50° E (south 50° east).

The angle is always measured from the north or south, and never from the east or west; and the letter $N$ or $S$ always precedes the angle, while $E$ or $W$ follows. If the line is in the direction $ON$, its bearing is said to be *due north;* similarly, the

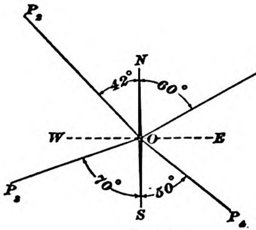


Fig. 3

bearings of the lines $OE$, $OS$, and $OW$ are *due east, due south,* and *due west*, respectively. Obviously, a bearing can never be greater than 90°. If $NS$ is the magnetic meridian, the bearings are *magnetic bearings*, while if $NS$ represents the true meridian, the bearings are *true bearings*.

**8. Angle Between Line and Meridian.**—Lines on the earth's surface are seldom horizontal; but when the direction

of a line is considered, the horizontal angle between the meridian and the line is understood. Let $A$ and $B$ in Fig. 4 be two points on the ground; $AN$, a horizontal line in the plane of the meridian

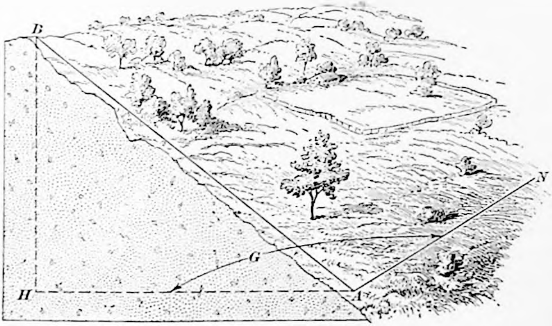


FIG. 4

at $A$; $BH$, a vertical line at $B$; and $AH$, a horizontal line that lies in the same vertical plane as $AB$ and intersects $BH$ at $H$. Then, the direction of the line $AB$ with respect to the meridian is represented by the angle $G$, of which both sides, $AN$ and $AH$, lie in the horizontal plane through $A$.

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### DESCRIPTION OF SURVEYORS' COMPASS

**9. General Construction.**—A surveyors' compass is shown in Fig. 5. Its essential parts are a magnetized steel bar, or *magnetic needle*, $a$, which is supported on a pivot $b$ at the center of a graduated circle $c$; and two sights $d$ attached to this circle.

The needle and the graduated circle are enclosed in a brass case $e$, called the *compass box*, which has a glass cover. The compass box is secured to a plate $f$, at the ends of which the sights $d$ are attached by means of milled-headed screws $g$ and $h$. The sights are brass bars with narrow vertical slits $i$ having small holes at the tops and bottoms; in sighting, a slit in one sight is viewed through a hole in the other sight. Often one of the slits in each sight is replaced by a wider opening with a very fine vertical wire to mark the line of sight. The sights

either are removable, as shown in Fig. 5, or are hinged so that they can be folded down over the compass box; thus, the instrument can be placed in a flat box when not in use. Sometimes one of the sights is graduated on the side so that angles in a vertical plane can be measured; in Fig. 5, the right-hand sight is so graduated.

In order to indicate when the plate *f* is horizontal, two *spirit levels j*, one parallel to the line of sight and the other at right angles to it, are screwed to the plate. Each level consists of a glass tube nearly filled with alcohol or a mixture of alcohol

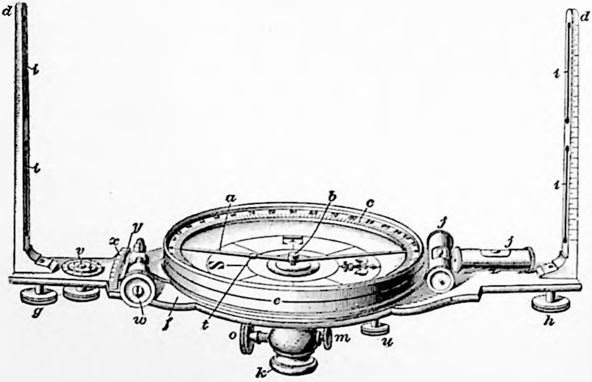


FIG. 5

and ether, the remaining space being occupied by a bubble of air. The tube is mounted in a brass case, which is attached to the plate.

**10. Tripod and Mounting.**—The compass is usually supported on a *tripod*, Fig. 6, which consists of three legs *a* shod with steel points and connected by hinge joints to a metal *tripod head b*. Occasionally a single straight pole about $4\frac{1}{2}$ feet long, called a *Jacob staff*, is used instead of a tripod. This staff has a pointed metal shoe which can be stuck in the ground. With both types of supports, a special mounting, such as that

shown in Fig. 7, is provided at the top for attaching the compass. The socket $k$, Fig. 5, which fits over the spindle $l$, Fig. 7, is either permanently attached or screwed to the plate $f$, Fig. 5, and is held in place on the spindle by the lock screw $m$, Fig. 5, which drops into the small groove $n$, Fig. 7. The socket turns freely on the spindle, but can be secured in any position by the clamp screw $o$, Fig. 5.

In Fig. 8 is shown a sectional view of the mounting with the socket $k$. On the lower end of the spindle $l$ is a carefully turned ball $p$; this rests in a spherical socket $q$ in the top of the sleeve $r$, which screws on the tripod head or staff head. The ball is held in the socket by a

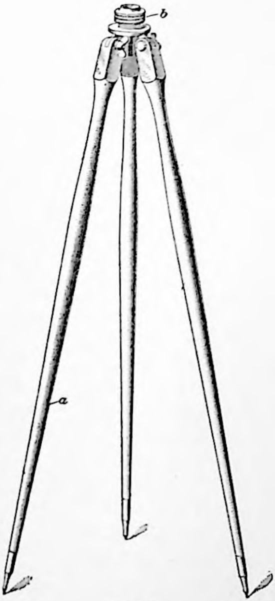


Fig. 6


Fig. 7

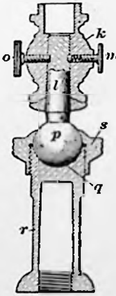


Fig. 8

ring $s$, which is screwed on the sleeve $r$ and which is ground to fit the ball. When the ring $s$ is loosened, the ball can be moved easily to level the instrument; and by tightening the ring, the ball can be held securely in the desired position. Such a joint is called a *ball-and-socket joint*.

Instead of the mounting shown in Fig. 7, an arrangement consisting of a plate and four leveling screws can be used for

leveling the compass. However, as the compass is never used in work requiring great accuracy, the simple ball-and-socket joint is generally employed.

**11. Needle.**—In Fig. 9 is shown a plan of the compass box. The needle *a* is mounted on a hard steel pivot *b*, ground to a fine point and secured in position at the center of the graduated circle *c*. The setting where the needle rests on the pivot usually consists of a jewel placed in a small metal cap on the needle. In the northern hemisphere, the south end of the needle carries a small sleeve or coil of wire *t*, Figs. 5 and 9. This coil serves to distinguish the north end from the south end, but the chief reason for its use is explained in the next article.

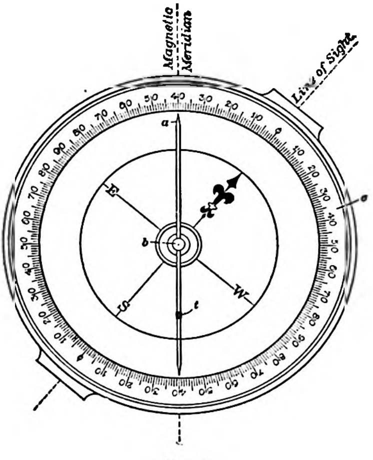


Fig. 9

By means of a lever, operated by the thumbscrew *u* shown on the under side of the compass plate in Fig. 5, the needle may be lifted off the pivot and pressed against the glass cover of the compass box. This prevents the pivot from becoming blunted when the compass is being carried.

**12. Dip of Needle.**—In the northern hemisphere, the needle is nearer to the north magnetic pole than to the south magnetic pole, and the north end of the needle is attracted more strongly than the south end. Therefore, when a needle is freely suspended, its north end is lower than the south end. The angle of inclination with the horizontal is called the *dip of the needle*.

In the southern hemisphere, the needle tends to dip with its south end lower than the north end. In order to keep the needle horizontal, the coil of wire previously mentioned is placed on the south end in the northern hemisphere and on the north end in the southern hemisphere. Since the dip of the needle varies with the latitude, the wire coil is made free to slide along the needle.

**13. Needle Circle.**—The graduated circle *c*, Figs. 5 and 9, is called the *needle circle* or *compass circle.* It is divided into quadrants by two lines, one in line with the sights and the other at right angles to the first. The graduations in line with the sights are marked 0; one of them is called the *north point*, and the other the *south point.* The sight near the north point is called the *north sight* and that near the south point is the *south sight.* The north point is sometimes indicated as in Figs. 5 and 9, but the letter *N*, or some other symbol, is often used; the south point is indicated by the letter *S*. The graduations at the other quadrant points are numbered 90, and are marked by the letters *E* and *W*, indicating east and west, respectively. The circle is sometimes divided into degrees, as shown here, but more often it is graduated in half degrees; each tenth degree is numbered.

The fact that the terms north and south are applied to the zero points does not mean that the line through these points is always in a north-and-south direction; these names are used for convenience in reading bearings, as will be explained later. For the same reason, the positions of the east and west points are interchanged. When one faces north, east is on the right and west on the left; but on the needle circle, *E* is placed on the left and *W* on the right.

**14. Outkeeper.**—The small dial *v*, Fig. 5, which is called an outkeeper, is used for counting tallies in chaining; it is turned by the milled-headed screw shown just beneath it. This is not an essential part of the instrument, and is not found on all compasses.

## ADJUSTMENTS OF COMPASS

**15. Conditions of Adjustment.**—A new compass, made by a good maker, is always in adjustment when it leaves the factory, but rough usage, a fall, or a hard blow may throw it out of adjustment. Besides several conditions that are taken care of in the construction of the instrument, the following are necessary for accurate work:

1. The bubbles should remain in the centers of the tubes throughout a complete revolution of the compass plate on the spindle.
2. The ends of the needle and the pivot must be in the same vertical plane.
3. The point of the pivot must be at the center of the needle circle.

To ascertain whether these conditions obtain, three tests are performed. If corrections are necessary, adjustments should be made. The methods of making these tests and adjustments will now be described.

**16. To Adjust Plate Levels.**—Set the tripod legs or the Jacob staff firmly in the ground, and bring the bubbles to the centers of the level tubes by moving the plate carefully by means of the ball-and-socket joint. Then revolve the compass on the spindle through 180°; that is, turn it end for end. If the bubbles remain in the centers of the tubes, the levels are in adjustment. But if turning the compass end for end causes either bubble to run toward one end of the tube, lower that end or raise the opposite end sufficiently to bring the bubble halfway back toward the center by means of the small screws that attach the ends of the tube to the plate. Then bring the bubbles to the centers by moving the plate by means of the joint. Repeat the operation until both bubbles remain in the centers of the tubes in both positions of the compass. This completes the first adjustment.

**17. To Straighten Needle.**—After the plate levels have been adjusted, bring the bubbles to the centers, release the needle, and, when it comes to rest, turn the compass so that the

north end of the needle is exactly opposite some prominent graduation mark of the needle circle; also, observe the exact reading of the south end of the needle. Then remove the glass and rotate the needle without turning the plate, so that the south end of the needle is exactly at the former reading of the north end. This can be done best by pushing the needle with a match or a small piece of wood, which does not attract the needle. With the needle in this position, observe the new reading of the north end. If the north end reads the same as the south end did before the needle was rotated, the needle is straight. If the first reading of the south end and the second reading of the north end are not the same, remove the needle from the pivot and bend the needle carefully to correct one-half the difference. Suppose that, in the first position, the north end of the compass is set at N 20° E and the south end reads S 19° 30′ W. Then when the south end is set at N 20° E, assume that the north end reads S 20° W. If the needle were straight, the north end in the second position would read S 19° 30′ W. To straighten the needle, make the north end read half-way between S 19° 30′ W and S 20° W, or S 19° 45′ W, when the south end reads N 20° E. Check the adjustment by repeating the operation, using different graduation marks.

It should be noticed that the two ends of the needle do not necessarily read alike for the same position of the needle; and if they do read alike, it does not indicate that the needle is straight. Thus, in the second position in the example just given, the reading of both ends of the needle is 20° but the needle was not straight. If in the second position the north end of the needle had read S 19° 30′ W to agree with the reading of the south end in the first position, no adjustment would have been necessary.

**18. To Center Needle Pivot.**—Having, if necessary, straightened the needle, turn the compass in several positions and observe if the readings of the north and south ends of the needle agree for each position. If they do, the point of the pivot is at the center of the needle circle. If the readings of the ends of the needle do not agree, find the position of the

plate which shows the greatest difference and clamp the plate securely. Determine the distance that one end of the needle would have to be moved to make the readings of the two ends alike. Then remove the needle from the pivot and bend the pivot carefully at right angles to the direction of the needle so that the point moves one-half that distance. Repeat the operations until the readings of the two ends of the needle agree in all positions of the plate.

## SURVEYING WITH A COMPASS

### FUNDAMENTAL PRINCIPLES

**19. Object of Compass Measurements.**—The compass is used primarily for measuring the magnetic bearings of lines. As previously explained, the angle between two lines may be determined from the bearings of the lines and, therefore, angles may be measured indirectly by means of the compass.

**20. Declination.**—Except in relatively few places, the magnetic meridian through a point on the earth's surface does not coincide with the true meridian at the point. In other words, the needle of the compass does not point toward the geographic poles of the earth. The angle between the magnetic meridian and the true meridian, or, what is the same thing, the angle that the compass needle makes with the true meridian, is called the *magnetic declination* or the *declination of the needle*. For some points on the earth, the north end of the needle is deflected west of true north, and for other points, it is east of true north. In the first case, the declination is said to be west, and in the second case, the declination is east, to correspond with the direction in which the north end of the needle is deflected from the true north. The amount of the declination of the needle is different in different localities, and also varies noticeably from year to year in the same locality. The differences and variations in the declination are not regular, though in a general way they follow a more or less definite system.

**21. Local Attraction.**—The compass needle may be deflected from the magnetic meridian by the attraction of an electric current or any near-by body of iron or steel, such as a pile of steel on the ground, the rails of a railway, a gas or water pipe, the tape or chain, keys or a knife on the observer, etc. Such a disturbing influence is called *local attraction*, although avoidable attractions caused by objects not fixed to the place are not usually included by this term.

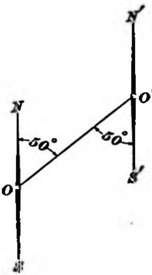

FIG. 10

**22. Forward Bearings and Back Bearings.**—In Fig. 10, $O$ and $O'$ are any two points and $NS$ and $N'S'$ are meridians at $O$ and $O'$, respectively. Since $NS$ and $N'S'$ are parallel, the angles $NOO'$ and $S'O'O$ are equal; thus, if the bearing of the line from $O$ to $O'$ is N 50° E, it follows that the bearing of the line from $O'$ to $O$ is S 50° W. The bearing of a line in one direction is called the *forward bearing;* while the bearing of the same line in the opposite direction is its *back bearing*. The angle is the same in the forward bearing and in the back bearing, but the quadrant is diagonally opposite. For example, if the forward bearing of a line is S 30° E, its back bearing is N 30° W. In every case, the back bearing is determined from the forward bearing by simply changing the letter $N$ to $S$ or $S$ to $N$, and by changing $E$ to $W$ or $W$ to $E$. The bearing of the line from $O$ to $O'$ is called the bearing of $OO'$; the bearing from $O'$ to $O$ is the bearing of $O'O$. Hence, in determining the bearing of a line, the order of the letters identifying the line is important.

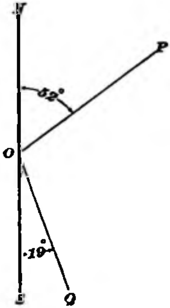

FIG. 11

**23. Angle Between Two Lines Whose Bearings Are Known.**—If an angle is turned from $OP$ to $OQ$, Fig. 11, the angle is called $POQ$; if the angle is turned from $OQ$ to $OP$, it is

$QOP$. Since it is possible to turn an angle either clockwise or counter-clockwise, it is important to state in which direction the angle is measured. Thus, the angle $POQ$ turned directly is called *$POQ$ clockwise*, or *$POQ$ to the right;* whereas, the angle $POQ$ turned through $N$ and $S$ is *$POQ$ counter-clockwise*, or *$POQ$ to the left.*

When two lines are in the same quadrant, that is, both bearings have the same letters, the angle between the lines is equal to the difference between the two bearings. For example, if the bearings of two lines are N 66° 30′ E and N 39° 15′ E, the angle between the lines is 66° 30′ − 39° 15′ = 27° 15′.

When the angle is required between two lines which are in different quadrants, it is usually advisable to make a rough diagram and to calculate the angle by inspection of the figure. First, a line is drawn to represent the meridian; then from any point on this line, the two given lines are drawn about in the directions indicated by the bearings. Thus, in Fig. 11, $NS$

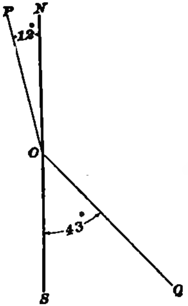


FIG. 12

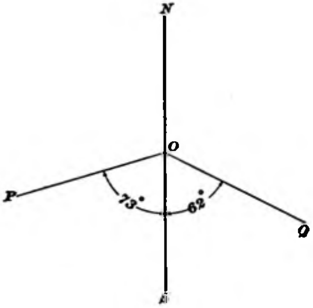


FIG. 13

represents the meridian, $N$ being the north point; the bearing of $OP$ is N 52° E; and that of $OQ$ is S 19° E. From the figure, it is seen that $POQ$ is equal to 180° − 52° − 19° = 109°. In Fig. 12, the bearing of $OP$ is N 12° W and the bearing of $OQ$ is S 43° E. Then angle $POS$ clockwise is 180° + 12° = 192°

and $POQ$ is $192°-43°=149°$. In Fig. 13, the bearing of $OP$ is S 73° W and that of $OQ$ is S 62° E; in this case, angle $POQ$ is obviously equal to $73°+62°=135°$.

In determining the angle between two lines from their bearings, it is important to take the bearing of each line away from

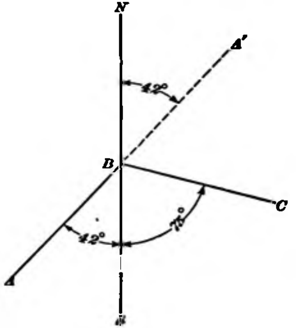

Fig. 14

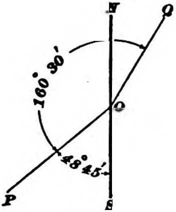

Fig. 15

the point of intersection of the lines. For example, suppose that in Fig. 14, $AB$ and $BC$ are two lines of a survey, the bearing of $AB$ being N 42° E and the bearing of $BC$ being S 75° E; let it be required to find the angle $ABC$. The desired angle is, in this case, the angle between $BA$ and $BC$, the bearing of $BA$ being equal to the back bearing of $AB$, or S 42° W. If the meridian $NS$ is drawn through $B$, it is seen that the angle $ABC$ is $42°+75°=117°$. If the bearing of $AB$ had been used, as shown by the dotted line $BA'$, the angle obtained would be that between $BC$ and $AB$ produced, or $A'BC$, which is equal to $180°-42°-75°=63°$.

**24. Bearing of Line From Bearing of Another Line and Angle Between Lines.**—When the angle between two lines and the bearing of one of the lines are known, the bearing of the other line may be found. As in the preceding article, it is advisable to make a rough diagram of the conditions. Suppose, for instance, that the bearing of the line $PO$, Fig. 15, is N 48°

45′ E and it is desired to find the bearing of another line $OQ$ making an angle $POQ$ equal to 160° 30′ clockwise. The bearing of $OP$ is equal to the back bearing of $PO$, or S 48° 45′ W. If $NS$ is the meridian through $O$, angle $NOP$ is 180° − 48° 45′ = 131° 15′, and $NOQ$ is 160° 30′ − 131° 15′ = 29° 15′. Hence, the bearing of $OQ$ is N 29° 15′ E.

If a given line $PO$, Fig. 16, has a bearing of N 30° 45′ E, and the angle between this line produced and another line $OQ$ is 70° 15′ to the right, the angle $NOQ$ is $NOC + COQ$ = 30° 45′ + 70° 15′ = 101°. Since this is greater than 90°, the line $OQ$ is in the southeast quadrant and the angle $SOQ$ is 180° − 101° = 79°. Hence, the bearing of $OQ$ is S 79° E. If the line $OP$,

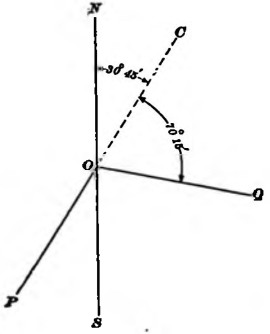


FIG. 16

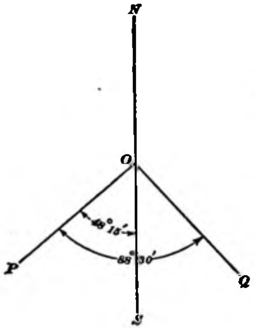


FIG. 17

Fig. 17, has a bearing of S 48° 15′ W and the angle between $OP$ and $OQ$ is 88° 30′ to the left, the angle $SOQ$ is 88° 30′ − 48° 15′ = 40° 15′ and the bearing of $OQ$ is S 40° 15′ E.

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#### EXAMPLES FOR PRACTICE

1. Find the angle between two lines $OA$ and $OB$, whose bearings are N 32° 15′ E and N 42° 30′ W. Ans. 74° 45′

2. What is the angle between lines $OA$ and $OB$, if the bearing of $OA$ is N 15° 30′ E and that of $OB$ is S 46° 45′ W? Ans. 148° 45′

3. The bearing of a line $AO$ is N 48° 15′ W and the bearing of $OB$ is N 76° 30′ W. Find the angle $A\ O\ B$. Ans. 151° 45′

4. The bearing of a line $OA$ is N 47° 30′ W. What is the bearing of a line $O\ B$ if angle $A\ O\ B$ is 138° 15′ to the left? Ans. S 5° 45′ E

5. If the bearing of a line $AO$ is S 20° 30′ W and the angle $AOB$ is 130° 45′ to the right, what is the bearing of $O\ B$? Ans. S 28° 45′ E

6. The bearing of line $A\ O$ is S 20° 15′ E. Find the bearing of the line $OB$, which makes an angle with $A\ O$ produced equal to 60° 45′ to the right. Ans. S 40° 30′ W

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## FIELD PROBLEMS

**25. Setting Up.**—In setting up a compass mounted on a Jacob staff, the staff is stuck as nearly as possible in a vertical position at the point over which the compass is to be set. If a tripod is used, the compass can be placed accurately by means of a plumb-bob suspended from a small ring directly under the center of the needle circle. However, in compass work, it is a waste of time to center the compass exactly over the station. With a little practice, it is possible to set up the compass almost over the point by judgment. If desired, the position of the compass can be determined by dropping a small pebble from below the center of the circle and observing how close to the marked point the pebble strikes.

After the compass is properly placed, the plate *f*, Fig. 5, is brought to a horizontal position, as shown by the spirit levels, by moving it on the ball-and-socket joint.

**26. Practical Suggestions.**—In leveling the compass by means of a ball-and-socket joint, do not grasp the sights, but take hold of the compass plate near the needle box.

When the compass is not in use or is being carried, keep the needle off the pivot point so that the point will not be dulled and the sensitiveness of the needle thus affected. After setting up in a new place, bring the needle as near as possible in the magnetic meridian before releasing it. Then the needle will come to rest more quickly and, also, the vibration being thus reduced, the sharpness of the pivot point will be preserved. In case the needle swings more than about 10° on each

side of the meridian, its progress can be checked by raising it off its pivot when near the center of its swing; when the needle is released again, the swing will be reduced. It is advisable to allow the needle to swing through about 3° before coming to rest, as it will then assume a more nearly correct position than if it is released very nearly in the meridian. When the compass is put away, or is allowed to remain in one position for a long time, permit the needle to assume its position in the meridian and then raise it off the pivot; if it is not in the meridian, it is liable to lose some of its magnetic strength.

**27. Taking Bearings.**—To find the bearing of a line, the compass is set up at some point on the line, preferably at one extremity. Then, a range pole is held vertically at some other point on the line, as far as possible from the compass. With his eye behind the south sight, the surveyor revolves the compass horizontally on the spindle until the range pole is approximately on the line through the sights. He then looks through the slits, his eye being at the south slit, and turns the plate until the range pole is exactly on the line of sight. The line of sight should be directed as nearly as possible to the bottom of the range pole in order to diminish the error due to any deviation of the pole from the vertical. The plate is then clamped in position, and the reading of the graduated circle opposite the north end of the needle is observed as explained in the following article. This is the bearing of the line.

In finding the bearing of an important line, it is always advisable to take a forward bearing and a back bearing. As previously stated, the angles should be equal. If the observed values agree closely, say within ½ degree or 1 degree according to the purpose of the survey, it is customary to take the average as the correct value. If the observed angles differ greatly, either the position of the needle was observed incorrectly or there was local attraction. The observations should be checked carefully; if no error is found, the cause of the difference must be local attraction. The method of correcting for local attraction will be treated later.

**28. Reading Needle Circle.**—In determining the bearing of a line, the plate is revolved until the sights are on the given line, with the north sight forward. The needle remains fixed in the magnetic meridian and, for magnetic bearings, the zero points of the needle circle are in the line of sight. Hence, the magnetic bearing of the line of sight is the angle between the needle and the line through the zero points of the needle circle. Let *OP*, Fig. 18 (*a*) or (*b*), be a line whose magnetic bearing is required. The compass is set at *O* and the sights are brought in the line *O P*. Then the angle between the

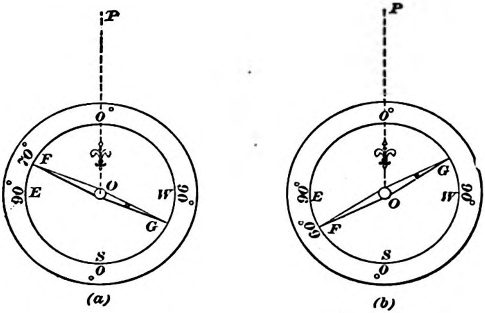


FIG. 18

needle *FG* (north end at *F*) and the line of sight, which is measured by the arc between the north end of the needle and a zero point of the circle, is the required bearing. If the bearing of *OP* is northeast, as in (*a*), or southeast, as in (*b*), the north end of the needle is to the left of the line of sight when the observer faces toward the north point of the needle circle; in other words, east bearings are read on the left half of the circle, where the letter *E* is placed. When the bearing of a line *O P* is northwest, as in Fig. 19 (*a*), or southwest, as in Fig. 19 (*b*), the north end of the needle is to the right of the line of sight; the letter *W* is, therefore, placed on the right side of the line through the south and north points of the circle.

When the letters are marked on the needle circle in this way, the quadrant in which a given line lies is indicated by the letters between which the north end of the needle rests. The angle between the meridian and the line is shown by the number of the graduation opposite the north end of the needle.

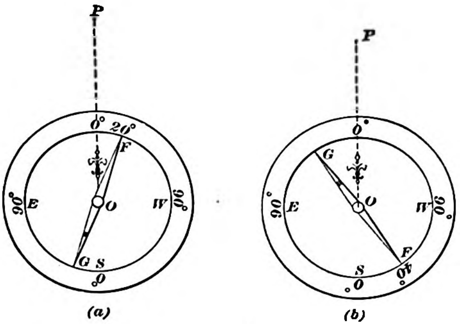


Fig. 19

Thus, in Fig. 18 (*a*), the bearing of the line *OP* is N 70° E because the north end of the needle is opposite the graduation marked 70 between the north and east points. In Fig. 18 (*b*), the bearing of *OP* is S 60° E; the bearing of *OP*, Fig. 19 (*a*), is N 20° W; and that of *OP* in Fig. 19 (*b*) is S 40° W.

Readings can be readily estimated to the nearest 10 minutes. It is a useless refinement to try to make readings closer than this with a compass because of the inaccuracy of the instrument. When the reading is taken, the eye of the observer should be exactly in line with the needle so that the point on the circle opposite the end of the needle may be determined correctly.

**29. East and West.**—The fact that the letters *E* and *W* are reversed on the needle circle of a compass often causes confusion to the beginner in laying off a line along a given bearing. In this latter case, it is incorrect to reverse east and west. A line whose bearing is northeast should be laid off to the right

of the meridian, as at $OP_1$ in Fig. 3, and not to the left; similarly, a line with a northwest bearing should have the general direction $O\ P_2$.

**30. Correcting for Local Attraction.**—Special care must be taken to avoid errors that may be caused by local attraction. Its presence may be detected best by comparing the forward and back bearings of the lines. If the needle readings indicate local attraction, the point of set-up at which it exists may be determined in the following manner: Suppose that the forward bearing from $O$ to $P$, Fig. 20, is N 85° 45′ E and the back bearing from $P$ to $O$ is S 75° 30′ W, local attraction thus being indicated. If $OP$ is a line of a survey, and $Q$ is another point on the survey, the forward and back bearings of $PQ$ are compared. If the bearings of $PQ$ and $QP$ agree, the local attraction must be at $O$; if the difference between the bearings of $PQ$ and $QP$ is the same as that between the bearings of $OP$ and $PO$, the local attraction is at $P$. In rare cases, there is local attraction at two of the points $O$, $P$, and $Q$. Then the

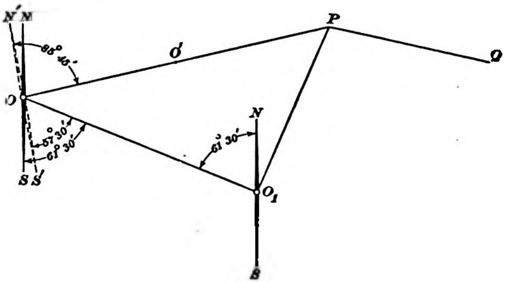

Fig. 20

survey is continued until the forward and back bearings of some line agree, and the other bearings are corrected from this line as described later in this article.

When $OP$ is not a line of a survey, it is often possible to determine the location of the attraction by setting up at a point, such as $O'$, on $OP$ about midway between $O$ and $P$;

then the bearing of $O'O$ or $O'P$ is taken. If it agrees with either the bearing of $OP$ or that of $PO$, it indicates the correct value and shows where the local attraction is. Thus, if the bearing of $O'O$ is S 85° 45′ W, the bearing of $OP$, previously determined, is correct and the local attraction is at $P$. If the bearing of $O'O$ is S 75° 30′ W, the bearing of $PO$ is correct and the local attraction is at $O$. When a slight difference is obtained, say less than 1°, it is customary to take the average of the two values as correct.

If it is not convenient to take an intermediate point on line $OP$, or if there is reason to suspect that there is local attraction all along $OP$, forward and back bearings to an outside point, as $O_1$, must be taken from both $O$ and $P$. If it is found that there is local attraction at both $O$ and $P$, the bearing of $OP$ must be corrected by determining the angle by which the needle is deflected by the local attraction. Suppose the bearing of $O_1O$ is N 61° 30′ W and that of $OO_1$ is, according to the reading of the compass, S 57° 30′ E. Let $NS$ represent the magnetic meridian through $O$ and $N'S'$ the position which the needle at $O$ assumes. If there is no local attraction at $O_1$, the angle $SOO_1$ is 61° 30′, and, therefore, the angle by which the needle is deflected is $61° 30' - 57° 30' = 4°$. As shown in the figure, the deflection is to the west of north. The value of angle $NOP$ is $85° 45' - 4° = 81° 45'$, and the correct magnetic bearing of $OP$ is N 81° 45′ E.

Attraction that is caused by something in the observer's clothing, as keys or a pocket-knife, can be detected by standing in one position to read the north end of the needle, and then going around to the other side of the compass to read the south end of the needle. If the two readings do not agree, there is an attraction on the observer.

**31. To Run Line Having a Given Bearing.**—It is frequently required to locate a line making a certain angle with another line whose bearing is known, or to relocate a line of an old survey. In the first case, the bearing of the required line can be calculated by the method explained in Art. 24; in the second case, the bearing of the line will be given. However,

in both problems, a line must be run from a given point in a certain direction.

The bearing of the required line having been determined, the compass is set over the known point on the line and the plate is rotated on the spindle until the north end of the needle indicates that bearing. The line of sight through the slits then has the desired direction, and, with his eye behind the south slit, the surveyor lines in a stake or pole through the north slit. This stake or pole is on the required line.

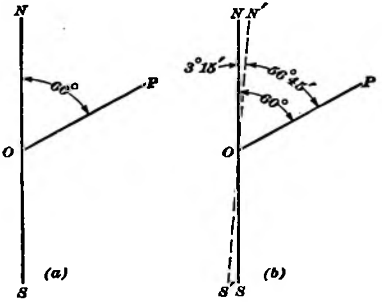


FIG. 21

Suppose, for example, that it is required to run a line $OP$, Fig. 21 ($a$), whose bearing is to be N 60° E. The compass is set at $O$ and the plate is rotated until the north end of the needle is opposite the graduation numbered 60 between the north and east points on the needle circle. A pole is then placed at $P$ in line with the sights.

As a check, the compass should be set over the new point and a back bearing taken. If the needle readings indicate local attraction, and it is found to be at the original point of set-up, the line must be relocated as follows: Suppose that a back bearing from $P$ to $O$, Fig. 21 ($a$), shows that there is local attraction. Suppose further that the attraction is found to be at $O$, and it deflects the needle 3° 15′ to the east. Let $NS$, Fig. 21 ($b$), be the magnetic meridian at $O$, and $N'S'$ the position taken by the needle. Then, in order that the correct bearing of $OP$ should be N 60° E, the line $OP$ should be run on a bearing of N 56° 45′ E, as indicated by the needle. If it is desired to lay off a given distance along the line, the stakes or pins at the end of each tape length can be lined in from the compass.

**32. Passing Obstacles.**—It is often required to determine the bearing of a line between two points, neither of which is visible from the other. Such a case is illustrated in Fig. 22, where the line of sight from $A$ to $B$ is obstructed by a building. With the compass set at $A$, locate point $C$ at any convenient distance and in any direction from $A$; record the length and bearing of $AC$. Then set up the compass at $B$ and run $BD$ with the same bearing and length as $AC$. Point $C$ should be so selected that $C$ and $D$ will be visible from each other. Set the compass at $C$ and determine the bearing of $CD$, which is equal to the bearing of $AB$. It is also permissible to set up at $D$ and take the bearing of $DC$, which is equal to the back bearing of $AB$.

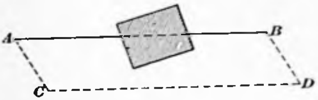

Fig. 22

To find the bearing of a fence or a wall, the compass is set up close to the wall, and a sight is taken on a pole held the same distance from the wall; then, the line of sight is parallel to the wall and has the same bearing.

**33.** In running a line in a given direction, obstacles are frequently encountered, as in Fig. 23, where $K$ and $L$ are buildings on the line $AB$. With the compass at $A$ and the line of sight in the proper direction, the point $P$ is set as near

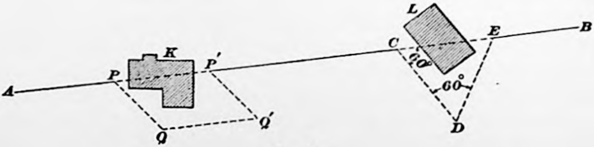

Fig. 23

to $K$ as possible. Then the compass is moved to $P$ and the line $PQ$ is run in any convenient direction; point $Q$ is marked on the ground and is so selected that $QQ'$ can be easily run parallel to $AB$. The length and bearing of $PQ$ are recorded. The compass is set at $Q$, the back bearing of $QP$ is

taken as a check, and the line $QQ'$ is run with a bearing equal to that of $AB$; the length of $QQ'$ should be such that $Q'P'$ can be run parallel to $PQ$. If the length of $AB$ is desired, the distance $QQ'$ is measured; otherwise it is not necessary. Then, the compass is set up at $Q'$, a back bearing is taken on $Q$ for a check, and the point $P'$ is so located that the bearing of $Q'P'$ is equal to the back bearing of $PQ$ and its length is equal to that of $PQ$. The point $P'$ is on line between $A$ and $B$, and the distance $PP'$ is equal to $QQ'$. If the compass is set up at $P'$, the line $AB$ can be continued in the proper direction by setting the sights so that the needle indicates the given bearing of $AB$.

**34.** The following method of passing the obstacle $L$, Fig. 23, is often more convenient than that just described. The line $AB$ having been run to $C$, near $L$, the line $CD$, making an angle of 60° with $AB$, is laid off; the bearing of $CD$ can be calculated by the method explained in Art. **24**. The distance $CD$ is so taken that a line from $D$, making an angle of 60° with $CD$, will clear the obstacle; the proper position of $D$ can be judged. The distance $CD$ having been measured, the compass is set up at $D$, and the point $E$ is so located that the line $DE$ makes an angle of 60° with $CD$ and the distance $DE$ is equal to $CD$. The point $E$, thus determined, is on $AB$, and the distance $CE$ is equal to $CD$. All bearings should be tested by back bearings.

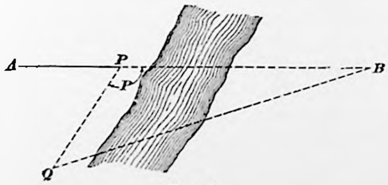

FIG. 24

**35.** In Fig. 24, the line $AB$ crosses a river; hence, its length cannot be measured directly nor be ascertained by either of the methods just explained. In this case, the bearing of $AB$ is determined and the distance from $A$ to a point $P$ near the bank is measured. Next, any other convenient point $Q$ is selected, and the length and bearing of $PQ$ are recorded;

theoretically $PQ$ may be of any length, but the results will be more accurate if $PQ$ is not less than about one-quarter of $PB$. Then the compass is set at $Q$, the bearing of $PQ$ is checked by a back bearing, and the bearing of $QB$ is observed. From the bearings of $AB$, $PQ$, and $QB$, the angles at $P$, $Q$, and $B$ can be calculated, as already explained; as a check, their sum should equal 180°. Then in the triangle $PQB$, the side $PQ$ is known, and the side $PB$ can be calculated by the relation

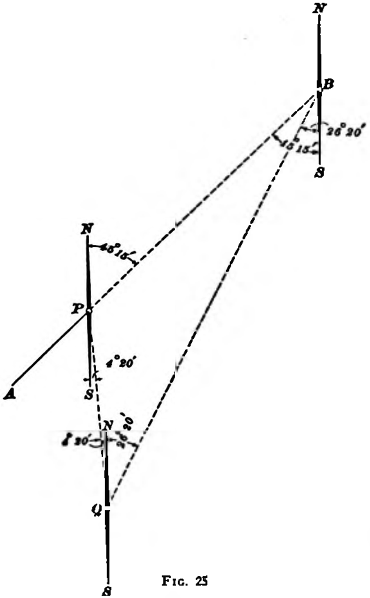


FIG. 25

$$PB = \frac{PQ \sin Q}{\sin B}$$

EXAMPLE.—Suppose that the length of $PQ$, Fig. 24, is 100 feet, the bearing of $AB$ is N 45° 15′ E, that of $PQ$ is S 4° 20′ E, and that of $QB$ is N 26° 20′ E. Find the distance from $P$ to $B$.

SOLUTION.—For convenience in determining the angles between the lines, Fig. 25 is drawn, in which $NS$ represents the meridian. The angle $BPQ$ is $180° - 4° 20' - 45° 15' = 130° 25'$; angle $PQB$ is $4° 20' + 26° 20' = 30° 40'$; and angle $PBQ$ is $45° 15' - 26° 20' = 18° 55'$. As a check, $130° 25' + 30° 40' + 18° 55' = 180°$. Then, $PB = \frac{100 \sin 30° 40'}{\sin 18° 55'} = 157.3$ ft. Ans.

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EXAMPLES FOR PRACTICE

1. If, in Fig. 24, the bearing of $AB$ is S 63° 15′ E, that of $PQ$ is S 43° 50′ W, that of $QB$ is N 84° 20′ E, and distance $PQ$ is 150 feet, what is the length of $PB$? Ans. 181.7 ft.

2. If, in Fig. 24, the bearing of $AB$ is N 45° 15′ W, that of $PQ$ is N 50° 30′ E, that of $QB$ is N 87° 25′ W, and distance $PQ$ is 200 feet, what is the length of $PB$? Ans. 199.7 ft.

3. The bearing of a line from $A$ to $B$ was measured as S 16° 30′ W. It was found that there was local attraction at both $A$ and $B$, and, therefore, a forward and a back bearing were taken between $A$ and a point $C$ at which there was no local attraction. If the bearing of $AC$ was S 30° 10′ E and that of $CA$ was N 28° 20′ W, what is the correct bearing of $AB$? Ans. S 18° 20′ W

NOTE.—In this and similar problems, a sketch is helpful.

## MAKING A COMPASS SURVEY

**36. Advantage of Compass.**—The compass is of great value in a survey where speed and economy are more important than accuracy and where small obstructions such as trees and rocks are frequently encountered along the line. At each obstruction, the compass can be moved to the opposite side and the line continued on the same bearing. Should the instrument as thus located be a foot or two off the correct line, no serious error will result since the new line is run parallel to the true position. It is assumed, of course, that local attraction is absent.

**37. Sources of Error.**—The chief causes of errors in obtaining the bearings of lines with a compass are local attraction, mistakes in reading the bearing, inaccuracy of the compass, poor adjustment, and variation in declination. The method of detecting and correcting for local attraction has been explained in a previous article. The adjustment of the instrument has also been described. Errors of observation due to poor sighting or incorrect reading of the needle can be eliminated only by exercising great care and by taking a forward and back bearing for each line. The compass is inaccurate because it is difficult to read the needle closely, and because different compass needles will not take exactly the same position under similar conditions and, therefore, will not indicate the same bearing for a given line. Variation in declination will be treated later.

**38. Survey Corps.**—The corps for making a compass survey should consist of at least three men, the compassman, and two

assistants who act sometimes as flagmen, and sometimes as chainmen. The number of men in the party and the duties of each depend on the purpose of the survey and the character of the country. If the country is thickly wooded, one or more axmen will be required to clear a narrow path in order to afford a fairly long sight for the compassman and to permit straight horizontal chaining. If the survey requires stakes

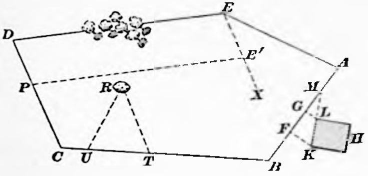

FIG. 26

at more or less regular intervals between the points at which the compass is set up, a stakeman is needed to cut, carry, and drive them. Stakes can be made of any well-seasoned wood, but in timbered country, it is often better to make the stakes from saplings as they are needed; the lower end is pointed to permit driving and the upper end is blazed for marking the station number.

**39. Surveying Established Lines.**—If it is desired to survey a series of lines between previously established points, such as the boundaries of the field in Fig. 26, the corners are first found and marked. The measurements sometimes consist only of the lengths and bearings of the boundary lines, but, often, it is also necessary to locate important objects.

A survey line, the length and bearing of which are determined, is commonly called a *course*. A system of connected courses is often called a *traverse*. If the end of the last course is also the beginning of the first course, the system is known as a *closed traverse*. Thus, the traverse shown in Fig. 26 from $A$ to $B$ to $C$ to $D$ to $E$ and back to $A$ is a closed traverse.

In the survey of a traverse, the compass is set up at each corner and the forward and back bearings of each course are taken. While the compass is directed along a side, that side is measured, in order that the compassman can line in the head chainman by looking through the slits. A point of set-up

may be either a solidly set stake, a mark in the root of a tree, an X cut in a rock, or any other suitable point. Marks on permanent objects should be fully described so that they can be found and identified at any time. If stakes are placed, they should be referenced by bearings and distances to prominent immovable objects near-by, so that they can be easily relocated in case they are destroyed.

To survey the boundary lines of the field shown in Fig. 26, the compass is set up at any corner, say $A$, and the bearing of $A\,B$ is observed. While the sight is directed along $A\,B$, its length is also measured. It is customary to take the back bearing of $EA$, which is the bearing of $AE$, at this time in order that it will not be necessary to set up again at $A$ at the end of the survey. The compass is next set at $B$; the bearing of $BA$, which is the back bearing of $AB$, is observed; and the bearing and the length of $BC$ are determined. Then the compass is set at $C$, from which point the bearings of $C\,B$ and $C\,D$ and the length of $C\,D$ are found. The operations are repeated at each corner if possible.

In case one corner is not visible from an adjacent corner, and it is not desirable to cut through the obstruction, the method of passing an obstacle, described in Art. 32, can be employed. Thus, suppose that the line of sight from $D$ to $E$, Fig. 26, is obstructed by many large trees; then, the length and the bearing of $PE'$, which are equal to those of $DE$, are determined instead. In this case, the point $P$ is taken on the line $D\,C$ for convenience, $EX$ is run on the same bearing as $D\,C$, and $EE'$ is made equal to $DP$.

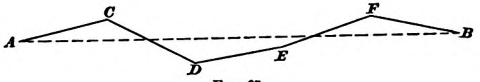

FIG. 27

**40. Random Traverse.**—It is often required to determine the length and the bearing of a line between two points that are a very great distance apart in wooded and rough country. The best method is to run a random traverse between the two points. For example, suppose that it is desired to find the

length and the bearing of the line between $A$ and $B$ in Fig. 27. The compass is set up over one end of the line, say $A$, and line $AC$ is run on a convenient bearing, as near as possible to that estimated for the line $AB$; the length and the bearing of $AC$ are recorded. Then the compass is moved to $C$, a back bearing is taken on $A$, and the forward bearing and the distance to a point $D$ are measured and recorded. Similarly, lines $DE$, $EF$, and $FB$ are run and their lengths and bearings determined. The method of computing the length and the bearing of $AB$ will be explained in another Section.

**41. Locating Objects.**—Important objects near the line of survey can be located in various ways, depending on circumstances. As explained in *Chain Surveying*, points can be located by distances along the line and perpendicular offsets; thus, the corners $K$ and $L$ of the house $H$ in Fig. 26 can be located by distances $BF$ and $FK$, and $BG$ and $GL$, respectively. Another method of locating a point is by taking the length and the bearing of the line from any point on the survey; in Fig. 26, the point $L$ can be located from $M$ by the length and the bearing of $ML$. Still another method, which is very convenient for locating inaccessible points, is to take the bearings of the lines to the point in question from any two points on the survey. In Fig. 26, the rock $R$ is located by the bearings of $UR$ and $TR$; if lines are drawn from $U$ and $T$ in the directions indicated by the bearings of $UR$ and $TR$, $R$ is at their intersection. Bearings to objects should be taken from corners of the field if possible in order to obviate an extra set-up.

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### FIELD NOTES FOR COMPASS SURVEY

**42. General Requirements.**—There are many methods of keeping the notes of a compass survey, each surveyor adopting such variations as seem best suited to his particular case. Only one method will be described here as an outline or a guide to illustrate the fundamental principles. The notes should be concise but in sufficient detail to be understood by any one familiar with the principles of surveying. All details

intended to be shown on the map should be noted, and nothing should be left to be supplied from the memory of the surveyor. The notebook commonly used, which is known as a *transit book*, has been described in *Chain Surveying*.

The notes should state whether the bearings are magnetic or true. If magnetic bearings are taken, the declination of the needle should be recorded, so that the true bearings can be calculated for future reference. All points of set-up should be fully described. Whenever possible the notes should be illustrated by a sketch. In some cases, all necessary information can be marked directly on the sketch.

The date of the survey and the names of the members of the party should always be given, because, in case of a lawsuit, all data regarding the survey may be required. As explained in *Chain Surveying*, the notes usually read upwards from the bottom of the page.

**43. Typical Compass Notes.**—In the form of notes shown in Fig. 28, the first column, headed *Course* or *Line*, gives the stations between which the courses are run; thus, $BC$ means the line between $B$ and $C$, with the set-up at $B$. In the second and third columns are the forward and back bearings; for the course marked $BC$, the forward bearing is from $B$ to $C$, and the back bearing is from $C$ to $B$ when the compass is set up at $C$. Since the compass was not set up at points *1* and *2*, no back bearings are recorded for *A1* and *B2*. The lengths of the courses are in the fourth column. The remainder of the left-hand page can be used for remarks, but generally these are made with the sketch on the right-hand page.

When the notes are examined, it will be noticed that the forward and back bearings of all the courses except $DE$ and $EF$ agree. This indicates that there was local attraction at $D$, $E$, or $F$. However, since the forward and back bearings of $CD$ agree, there can be no local attraction at $D$. Similarly, the forward and back bearings of $FA$ show that there is no local attraction at $F$. Then the correct bearings of $DE$ and $EF$ are, respectively, S 12° 10′ W and S 74° 40′ W. The conditions are shown in Fig. 29; $NS$ represents the magnetic merid-

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SURVEY OF HILL FARM
near Vincennes, Ind.
All bearings are magnetic.
Declination 2° East.

July 20, 1925
James Wheeler, Surv.
Frank Wilson
George Roberts } Ass'ts.
John Black

| Course | Bearings Forward | Bearings Back | Dist. Feet | | | Remarks |
|---|---|---|---|---|---|---|
| F A | N 4°30'W | S 4°30'E | 250.0 | | | F is monument at S.E cor. of Meadowbrook Farm |
| E F | S 74°40'W ~~S 76°00'W~~ | N 74°40'E | 374.9 | | | E is notch in root of sycamore tree |
| D 5 | | | 50 | | | |
| D E | S 12°10'W | N 12°10'E ~~N 13°30'E~~ | 275.0 | | | D is stake Reference-maple tree S 21°E – 31 ft. |
| C 4 | | | 120 | | | |
| C D | S 21°50'E | N 21°50'W | 193.8 | | | C is notch in root of large oak stump |
| B 3 | | | 125 | | | |
| B 2 | S 22°E | | 228 | | | |
| B C | N 80°20'E | S 80°20'W | 220.6 | | | B is stake. Reference – beech tree N45°W – 63 ft. |
| A 1 | Due E | | 251 | | | |
| A B | N 30°20'E | S 30°20'W | 300.0 | | | A is monument at N.E. cor. of Meadowbrook Farm |

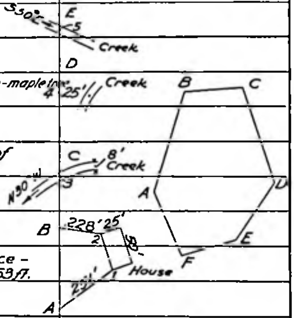



FIG. 28

ian at $E$ and $N'S'$ is the actual position of the needle. Each of the angles $N'EN$ and $S'ES$ is equal to 1° 20′. The corrected bearings are written above the observed bearings as shown in the notes; the incorrect observed bearing should not be erased. Sometimes, one of the vacant columns is used for the corrected magnetic bearings, and the other column is used for the true bearings, which may be calculated from the magnetic bearings by correcting for the declination by the method which will be explained later. For the courses forming the boundaries of the field, the bearings are read as accurately as possible, usually to the nearest 10 minutes or quarter-degree, but for the lines locating points 1, 2, etc., the bearings are taken only to the nearest degree.

The meaning of the figures in the column of distances is obvious. The length of course $AB$ is 300.0 feet and that of $EF$ is 374.9 feet. The lengths of the boundaries of the field are given to the nearest tenth of a foot; but for the distances to other points, as 1, 2, etc., the nearest foot is accurate enough.

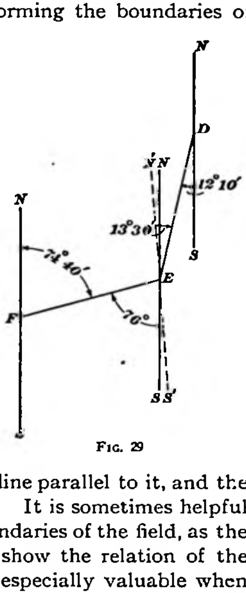


FIG. 29

As in the notes for a chain survey, the survey line is usually represented by the red line at the center of the right-hand page, or a line parallel to it, and the notes are supplemented by a sketch. It is sometimes helpful to make a rough diagram of the boundaries of the field, as the polygon $ABCDEF$ in Fig. 28, to show the relation of the various lines. Such a diagram is especially valuable when the main survey lines form a complicated system.

**44. Sketching Method.**—In some cases, the sketch is made so complete that additional notes are not necessary. Thus, the sketch for the notes just given may be made as shown in Fig. 30. The title of the survey, the date, and the names of

the members of the party should be given as in the written notes. The sketch is not necessarily drawn to scale, but is intended to give only an idea of the shape of the farm and the location of the house and creek. On a map the declination is

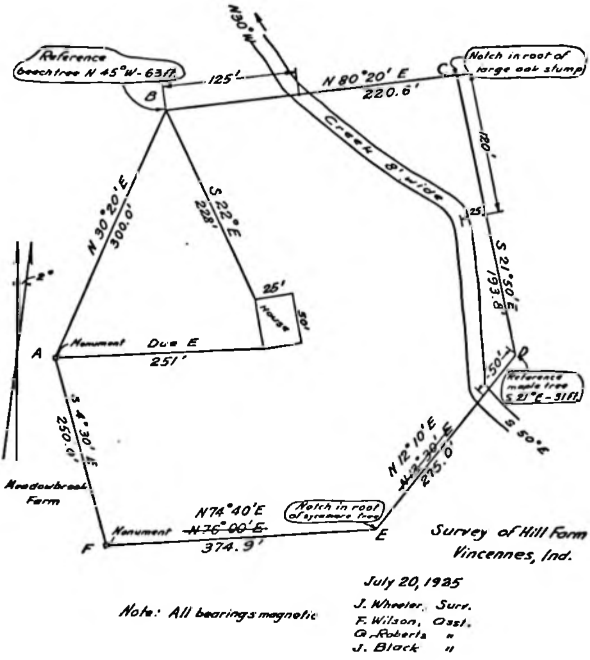



FIG. 30

usually shown by drawing two lines to represent the true and magnetic meridians and marking the angle between them, as shown in Fig. 30. Usually, the true meridian has a full arrowhead at the north end, and the magnetic meridian is indicated by half of an arrowhead at its north end.

In writing bearings along lines, it is customary to consider the direction of the line as that in which the bearing is read. For instance, in Fig. 30, the bearing is read from $A$ toward $F$ and is, therefore, written S 4° 30′ E, although it is observed from $F$ to $A$ as N 4° 30′ W. If it is desired to write all bearings as they are first observed, that is, as they would be given as forward bearings in written notes, a small arrow is used to indicate reversals. Thus, the bearing of the creek at the upper end is read in a southeasterly direction; since it is written N 30° W, the direction corresponding to the written bearing is indicated by the small arrow. When the back bearing does not agree with the forward bearing, both values are written on the line, as shown for courses $DE$ and $EF$. Then the incorrect value is crossed out, but not erased.

Keeping the notes by a sketch alone is often impracticable, because it is necessary to get much information in the small space on an ordinary notebook page; therefore, it is often impossible to make the figures legible and to show clearly to what the dimensions refer. The combination of written notes and sketches is usually best.

## DECLINATION

**45. Determination of Declination.**—In nearly all large cities and in many county seats, the direction of the true meridian has been established by astronomical methods and marked by permanent monuments. The angle between this line and the compass needle gives the declination on the date of observation. For ordinary purposes, the declination at any point in the United States can be found from charts which are published at intervals by the United States Coast and Geodetic Survey, and which can be secured from the U. S. Coast and Geodetic Survey, Washington, D. C.

On these charts are lines, called *isogonic lines*, which connect all points of equal declination; these lines are drawn for each degree of declination. The line of zero declination is called the *agonic line*. The locations of isogonic lines vary continually, but the charts give the amount of this variation in a year so

that the approximate declination can be found for any time within several years after the date on the chart. The declinations at points between isogonic lines can be found by interpolation, it being assumed that the declination varies uniformly from line to line.

**46. Variation in Declination.**—As previously explained, the magnetic meridian at any locality is constantly changing, the total rate of change at any time being due to several independent variations.

First, there is a slow change in the direction of the magnetic meridian, which returns to its original position after several hundred years. This shifting is called the *secular variation*, and, as it is practically periodic, its amount can be determined closely for any date and locality. Values are given on the charts of the United States Coast and Geodetic Survey, which have been previously mentioned.

In addition, there is a change during the day, called the *diurnal variation*. The needle swings back and forth once daily through an arc whose value depends on the locality and on the time of year; in the United States, the total swing varies from about 3 minutes in winter to 12 minutes in summer. The needle reaches the center of its swing at about 11 A. M. and 6 P. M.

Other variations which cannot be predicted are caused by magnetic disturbances in the air. Such disturbances are called *magnetic storms*, and sometimes cause deflections of more than ½ degree. Their presence is indicated by rapid fluctuations of the needle, and observations should not be taken at such a time.

**47. Relation Between True and Magnetic Bearings.** From the definition of declination, it is evident that the difference between the true and magnetic bearings of a line is equal to the declination of the needle for the locality. Hence, if the declination of the needle is known, the true bearing of a line can be found from its magnetic bearing, and vice versa. Thus, suppose the declination is 2° 18′ east, and the magnetic bearing of a line $OX$ is N 43° E; let it be required to find

the true bearing of the line $OX$. If $NS$, Fig. 31, is the true meridian and $N'S'$ is the magnetic meridian, angle $NON'$ is 2° 18′ to the right. Since angle $N'OX$ is 43°, angle $NOX$ is 43°+2° 18′=45° 18′, and the true bearing of $OX$ is N 45° 18′ E. Suppose it is desired to find the true bearing of a line $OA$, whose magnetic bearing is S 89° 15′ W, when the declination is 2° 18′ east. In Fig. 31, angle $S'OA$ is 89° 15′ and angle $SOA$ is 89° 15′+2° 18′=91° 33′. Since this is greater than 90°, the line $OA$ lies in the northwest quadrant with respect to the true meridian, and the angle $NOA$ is 180°$-SOA$=180° $-$91° 33′=88° 27′. Hence, the true bearing of $OA$ is N 88° 27′

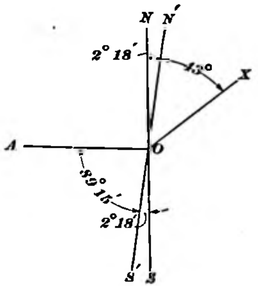


Fig. 31

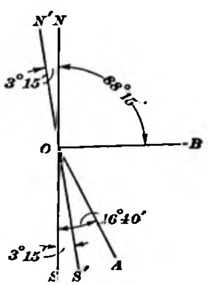


Fig. 32

W. It is important to remember that the line is represented but once whereas there are two meridians.

48. Often the true bearings of lines are given on maps; but to relocate one of these lines with a compass, it is required to find its magnetic bearing. For example, suppose that it is desired to obtain the magnetic bearing of a line $OA$, Fig. 32, whose true bearing is S 16° 40′ E, when the declination is 3° 15′ west. Here, $NS$ is the true meridian and $N'S'$ is the magnetic meridian. Then angle $SOA$ is 16° 40′, and $S'OA$ is 16° 40′ $-$3° 15′=13° 25′. The magnetic bearing of $OA$ is, therefore, S 13° 25′ E. Similarly, if the true bearing of $OB$ is N 88° 15′ E, angle $N'OB$ is 88° 15′+3° 15′=91° 30′. Since this is greater than 90°, the line $OB$ is in the southeast quadrant with respect

to the magnetic meridian. Thus, angle $S'OB$ is $180° - 91° 30' = 88° 30'$, and the magnetic bearing of $OB$ is S 88° 30′ E.

**49. Declination Arc.**—In the foregoing explanations, it was assumed that the line through the zero marks of the needle circle coincided with the line through the sights. For this position of the zero points, the needle readings give magnetic bearings. True bearings may be shown by the

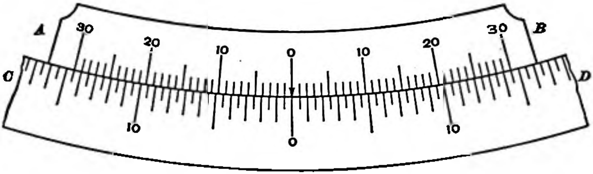



FIG. 33

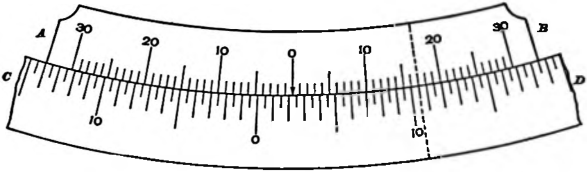
needle, however, if the line through the zero points is rotated out of the line of sight by an amount equal to the declination. For the purpose of rotating the needle circle with respect to the line through the sights, the screw $w$, Fig. 5, is provided. The amount of rotation is measured on a graduated scale $x$ with the aid of a vernier $y$.

A *vernier* is an auxiliary scale by means of which a main scale can be read more accurately. The zero of the vernier $y$ is in line with the north and south points of the needle circle, and the vernier rotates with the needle circle. The zero of the scale $x$ is on the line between the slits in the sights, that is, it is on the line of sight. and remains fixed as the vernier rotates.

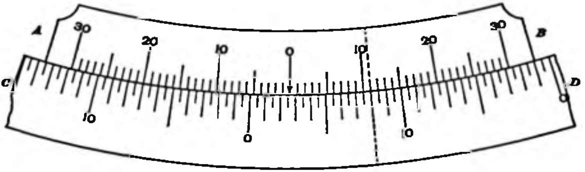
If the magnetic bearing of a line is wanted, the zero of the vernier $y$, Fig. 5, should coincide with the zero of the scale $x$. If the true bearing of a line is required, it can be read directly on the needle circle by setting the zero of the vernier at a point on the scale corresponding to the declination of the needle. Therefore, the combination of the graduated scale $x$ and the vernier $y$ is called the *declination arc* or *declination vernier*.

Both the scale and the vernier are graduated on each side of

zero, as shown in Fig. 33, where $A\,B$ is the vernier and $C\,D$ is the scale. If the line of sight is in the true meridian and the declination arc reads zero, the north point of the needle is east of the zero of the needle circle for east declination and west of zero for west declination. If the position of the needle is to indicate true bearings, the needle circle must be rotated to read zero when the line of sight is in the true meridian. To obtain this condition, the needle circle and the vernier, which moves with it, must be rotated in a clockwise direction when the declination is east and in a counter-clockwise direction when the declination is west. Thus, if the declination is east, the zero of the vernier in Fig. 33 is moved toward $C$, or clockwise; and if the declination is west, the vernier is moved toward $D$. In setting the declination, that side of the vernier is used on




(a)




(b)

FIG. 34

which the numbers increase in the direction in which the vernier moves. When the vernier is moved toward $C$ in Fig. 33, the part of the vernier between 0 and $A$ is used; if the vernier moves toward $D$, the part between 0 and $B$ is used.

**50.** The principle on which the construction of a vernier is based is fully explained in another Section. For the purpose

of reading and setting the declination arc, the following outline is sufficient. In Fig. 33, each division on the scale *CD* represents half of a degree, or 30 minutes. Therefore, for any position of the vernier, the reading to the next smaller half-degree is given by the number of the scale graduation just preceding the zero of the vernier, that is, between the zero of the scale and the zero of the vernier. To this value is added a number of minutes equal to the number of the vernier graduation mark that coincides with a graduation of the scale (it is not necessary to observe which scale graduation).

Suppose, for example, that the zero of the vernier is set as shown in Fig. 34 (*a*). The scale graduation nearest the zero of the vernier and between the zero of the vernier and the zero of the scale is 2°; the seventeenth graduation of the right part of the vernier coincides with a scale graduation. Hence, the reading is 2° 17′. In Fig. 34 (*b*), the scale graduation preceding the zero of the vernier represents 2° 30′, and the eleventh vernier mark coincides with a scale graduation. The reading. therefore, is $2° 30' + 11' = 2° 41'$.

To set the vernier at 3° 13′, put the zero of the vernier between the scale graduations representing 3° and 3° 30′, and then bring the thirteenth division of the proper half of the vernier opposite some graduation on the scale. To set the vernier at 4° 38′, place the zero of the vernier between the graduations of 4° 30′ and 5°, and bring the eighth mark of the vernier to coincide with some graduation on the scale.

**51. Rerunning Old Surveys.**—It is sometimes necessary for a surveyor to retrace the boundaries of a tract of land from bearings and distances given in original land warrants or very old deeds or maps. If the true bearings are given, or if the declination of the needle at the time of the original survey is known, the present magnetic bearings can be readily determined. If the old magnetic bearings are given but the declination is not known, it is sometimes possible to locate one or more of the original lines by means of legally recognized corners, and to measure the true bearings. By comparing the true bearings with the old bearings, the declination can be determined, and

the true bearings of the other lines can be readily found from the given original magnetic bearings. The old lines can then be rerun from their true bearings or from their present magnetic bearings. Sometimes, however, there are inaccuracies in the old surveys. In such cases, the old corners or boundaries cannot be altered or moved. Adjustments can be made only by mutual agreement between the interested parties, or by a court decision.

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**EXAMPLES FOR PRACTICE**

1. The declination is 2° 10′ west. Find the true bearings of the lines whose magnetic bearings are (*a*) N 48° 50′ E; (*b*) S 1° W.

Ans. { (*a*) N 46° 40′ E; (*b*) S 1° 10′ E

2. The declination is 4° 15′ east. Find the true bearings of the lines whose magnetic bearings are (*a*) N 87° 10′ E; (*b*) S 3° E.

Ans. { (*a*) S 88° 35′ E; (*b*) S 1° 15′ W

3. The declination is 1° 5′ east. Find the magnetic bearings of the lines whose true bearings are (*a*) S 88° 55′ E; (*b*) S 47° 10′ W.

Ans. { (*a*) Due east; (*b*) S 46° 5′ W

4. The declination is 3° 25′ west. Find the magnetic bearings of the lines whose true bearings are (*a*) S 88° 55′ W; (*b*) S 1° E.

Ans. { (*a*) N 87° 40′ W; (*b*) S 2° 25′ W